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THE ASSOCIATION was organized in Boston, Mass., on June 21, 1882, with the object of providing its members with means of social intercourse and for the exchange of knowledge pertaining to the construction and management of water works. From an original membership of only TWENTY-SEVEN, its growth has prospered until now it includes the names of 800 men. Its membership is divided into six classes, viz:

A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

An Honorary Member shall be a person of acknowledged eminence in some branch of water supply or of engineering.

A Life Member shall be a member whose service to the Association entitles him to special recognition by the Association.

A Junior shall be not less than eighteen years nor more than twenty-five years of age, a student or connected with water supply work.

An Associate shall be either a person, firm or corporation engaged in manufacturing or furnishing materials or supplies for the construction or maintenance of water works.

A Corporate Member shall be either a Water Board, Commission, Company or Municipal Corporation.

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Associates	10.00	Associates	20.00
Corporate Members	10.00	Corporate Members	10.00

This Association has six regular meetings each year, all of which, except the annual convention in September, are held at Boston.

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STEPHEN H. TAYLOR.

President of the New England Water Works Association.
1924–1925.

New England Water Works Association

ORGANIZED 1882.

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This Association, as a body, is not responsible for the statements or opinions of any of its members.

SOME FLOODS AND DROUGHTS ON NEW ENGLAND STREAMS.

BY CALEB MILLS SAVILLE.*

[*Read October 1, 1924.*]

Southern New England, at least, recently has passed through a period of drought (1923) so severe that many water supply systems have been seriously affected. This district also has experienced within a year two periods of excessive watershed run-off following each other consecutively. One of these was the greatest of record at least during the past half-century. During the past 10 years the hydrological cycle has passed from crest to hollow and exhibited features which are of more than passing interest to water supply workers. The details of this period are shown in Table 1.

For the calendar year 1920 the rainfall on the Nepaug River watershed of the Hartford (Conn.) water supply system was 51.78 in., or 16% above the station average of 11 years, with a run-off of 33.92 in., which was nearly 32% above the average of that area.

The calendar year 1922, with a rainfall of 43.87 in., although only less than the station average by 2%, had an average run-off that was the lowest in 11 years, being only 20.37 in., or 16% below the station average. In 1921 the rainfall was less than that of 1922 by 3.52 in. (40.35), yet the run-off was 21.29 in., or 0.92 in. more than in the following year.

The lowest rainfall was that of 1916, which was 37.68 in. This was 6.92 in., or 15.5% below the watershed average. The run-off of this year was 20.99 in., which is 3.22 in. below the average, but is 0.62 in. in excess of the run-off of 1922, which was the lowest of record for the Nepaug area.

Similar conditions appeared in nearby streams. On the Manhan River, the year 1920 produced the highest run-off experienced in the 28 years of record,† while that for 1921 was among the lowest that have been recorded.

It is the purpose of this paper to bring to attention some of the elements that have contributed to these conditions as they have been observed on the watersheds of the Hartford Water Board and others in that vicinity.

* Manager and Chief Engineer, Hartford (Conn.) Water Department.

† Courtesy of Mr. P. J. Lucey, Engineer, Holyoke Water Department.

TABLE 1.

Rainfall and Run-off of Nepaug River Watershed.

Year.	January.			February.			March.			April.			May.			June.			July.		
	Rainfall, Inches.	Run-off, Inches.	% Rainfall Collected.	Rainfall, Inches.	Run-off, Inches.	% Rainfall Collected.	Rainfall, Inches.	Run-off, Inches.	% Rainfall Collected.	Rainfall, Inches.	Run-off, Inches.	% Rainfall Collected.	Rainfall, Inches.	Run-off, Inches.	% Rainfall Collected.	Rainfall, Inches.	Run-off, Inches.	% Rainfall Collected.	Rainfall, Inches.	Run-off, Inches.	% Rainfall Collected.
1912	...	...	...	...	...	...	...	...	...	4.36	4.85	111.2	4.22	3.54	83.8	0.80	0.84	105.0	1.89	0.31	16.4
1913	3.46	3.10	89.7	2.23	1.50	67.3	5.62	4.40	78.4	4.93	3.71	75.3	4.19	2.20	52.5	0.87	0.67	76.9	1.85	0.27	14.6
1914	3.15	2.20	69.8	2.19	1.95	89.0	4.91	5.33	108.5	4.29	4.33	101.0	3.34	2.98	89.2	3.26	0.83	25.5	4.25	0.70	16.5
1915	6.78	4.80	70.8	4.55	4.29	94.4	0.13	1.28	985.0	2.05	1.74	84.9	2.45	1.29	52.7	2.66	0.60	22.6	6.49	1.07	16.5
1916	1.13	2.89	255.8	4.36	3.30	75.7	2.44	2.81	115.2	2.20	4.12	187.3	3.21	1.95	60.7	4.65	1.61	34.6	4.35	1.01	23.2
1917	3.72	2.26	60.8	2.47	0.69	28.0	3.90	5.74	147.2	2.61	3.52	134.9	3.94	2.27	57.6	6.33	3.05	48.2	2.77	0.61	22.0
1918	3.25	0.92	28.0	2.75	3.02	109.8	2.80	4.70	167.8	3.60	2.99	83.1	5.00	2.82	56.4	3.00	1.17	39.0	4.50	0.61	13.5
1919	2.67	2.39	89.5	3.23	1.28	39.6	6.51	5.52	84.8	2.97	4.01	135.0	7.55	4.31	57.1	1.88	0.96	51.0	3.27	0.70	21.4
1920	2.59	0.78	30.1	4.35	1.50	34.5	4.19	8.39	200.0	4.02	5.51	137.3	3.47	2.99	86.2	6.47	2.72	42.0	4.88	1.99	40.8
1921	2.73	2.41	88.3	3.07	1.52	49.5	2.98	5.18	173.8	5.70	4.03	70.0	2.74	2.18	79.6	3.76	0.81	21.5	4.15	1.31	31.6
1922	1.78	0.52	29.2	2.02	1.07	53.0	4.30	4.52	105.1	2.48	3.85	155.2	5.94	3.27	55.1	5.03	1.39	27.6	4.36	1.06	24.3
1923	7.75	2.84	36.6	1.66	1.20	72.3	2.79	5.80	207.9	4.68	5.14	109.8	2.58	2.71	105.0	3.83	1.12	29.2	1.73	0.35	20.2
Mean	3.55*	2.28*	64.2	2.99*	1.94*	64.9	3.69*	4.88*	132.2	3.66	3.98	108.7	4.05	2.71	66.9	3.55	1.31	36.9	3.71	0.83	22.4

TABLE 1 — (*Continued*).

Year.	August.			September.			October.			November.			December.			For the Year.		
	Rainfall, Inches.	Run-off, Inches.	% Rainfall Collected.	Rainfall, Inches.	Run-off, Inches.	% Rainfall Collected.	Rainfall, Inches.	Run-off, Inches.	% Rainfall Collected.	Rainfall, Inches.	Run-off, Inches.	% Rainfall Collected.	Rainfall, Inches.	Run-off, Inches.	% Rainfall Collected.	Rainfall, Inches.	Run-off, Inches.	% Rainfall Collected.
1912	3.87	0.39	10.1	3.10	0.42	13.5	4.51	0.79	17.7	4.78	1.58	33.0	4.31	1.94	4.5	...	...	...
1913	4.91	0.32	6.5	4.84	0.65	13.4	10.82	4.06	37.6	3.78	2.74	72.4	2.74	2.12	77.4	50.24	25.74	51.3
1914	2.62	0.42	16.0	0.32	0.23	71.9	3.58	0.52	14.4	3.47	0.68	19.5	4.00	0.74	18.5	39.38	20.91	53.1
1915	8.63	2.65	30.7	3.50	1.14	32.6	3.08	1.35	43.8	3.43	1.39	40.5	5.30	2.65	50.0	49.05	24.25	49.5
1916	2.94	0.45	15.3	4.93	0.62	12.6	1.54	0.55	35.7	3.10	0.83	26.8	2.83	0.85	30.0	37.68	20.99	55.7
1917	5.12	0.80	15.6	0.67	0.32	47.8	8.35	2.87	34.4	1.10	1.73	157.3	3.17	1.27	40.1	44.15	25.13	56.9
1918	2.60	0.50	19.3	6.90	0.90	13.0	1.00	0.77	77.0	2.80	1.27	45.2	3.50	2.23	63.7	41.70	21.90	52.5
1919	3.82	0.56	14.7	5.49	1.18	21.5	2.79	0.88	31.5	4.91	2.38	48.4	2.22	2.24	100.1	47.31	26.41	55.8
1920	3.39	0.93	27.4	7.86	0.93	11.8	0.66	1.79	271.2	4.63	2.24	48.4	5.27	4.15	78.7	51.78	33.92	65.5
1921	2.54	0.47	18.5	3.96	0.33	8.3	1.50	0.44	29.3	5.08	1.16	22.8	2.14	1.45	67.7	40.35	21.29	52.7
1922	5.49	1.04	19.0	2.04	0.78	38.2	5.15	1.37	26.6	1.30	0.81	62.3	3.98	0.69	17.3	43.87	20.37	46.4
1923	2.46	0.30	12.2	3.86	0.40	10.4	6.44	1.42	22.0	2.68	1.14	42.5	4.61	2.94	63.8	45.07	25.36	56.3
Mean	4.03	0.74	18.4	3.96	0.66	16.7	4.12	1.40	34.0	3.42	1.49	43.6	3.67	1.94	52.9	44.60*	24.21*	54.3

Drainage Area: 26.8 sq. mi., 1912 to 1917; 22.8 sq. mi. in 1917; 23.9 sq. mi. since 1917.
Rainfall: For 1912, average of Barnes, Bakersville and Douglas; 1913, 1914 and 1917, average Barnes and Bakersville; 1915–1916, average Barnes, Bakersville and Lyman; 1918, see — 1918, average Bakersville-Nepaug Dam.

* 11 years only.

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1912	...	...	...	...	...	...	...	...	...	4.36	4.85	111.2	4.22	3.54	83.8	0.80	0.84	105.0	1.89	0.31	16.4
1913	3.46	3.10	89.7	2.23	1.50	67.3	5.62	4.40	78.4	4.93	3.71	75.3	4.19	2.20	52.5	0.87	0.67	76.9	1.85	0.27	14.6
1914	3.15	2.20	69.8	2.19	1.95	89.0	4.91	5.33	108.5	4.29	4.33	101.0	3.34	2.98	89.2	3.26	0.83	25.5	4.25	0.70	16.5
1915	6.78	4.80	70.8	4.55	4.29	94.4	0.13	1.28	985.0	2.05	1.74	84.9	2.45	1.29	52.7	2.66	0.60	22.6	6.49	1.07	16.5
1916	1.13	2.89	255.8	4.36	3.30	75.7	2.44	2.81	115.2	2.20	4.12	187.3	3.21	1.95	60.7	4.65	1.61	34.6	4.35	1.01	23.2
1917	3.72	2.26	60.8	2.47	0.69	28.0	3.90	5.74	147.2	2.61	3.52	134.9	3.94	2.27	57.6	6.33	3.05	48.2	2.77	0.61	22.0
1918	3.25	0.92	28.0	2.75	3.02	109.8	2.80	4.70	167.8	3.60	2.99	83.1	5.00	2.82	56.4	3.00	1.17	39.0	4.50	0.61	13.5
1919	2.67	2.39	89.5	3.23	1.28	39.6	6.51	5.52	84.8	2.97	4.01	135.0	7.55	4.31	57.1	1.88	0.96	51.0	3.27	0.70	21.4
1920	2.59	0.78	30.1	4.35	1.50	34.5	4.19	8.39	200.0	4.02	5.51	137.3	3.47	2.99	86.2	6.47	2.72	42.0	4.88	1.99	40.8
1921	2.73	2.41	88.3	3.07	1.52	49.5	2.98	5.18	173.8	5.70	4.03	70.0	2.74	2.18	79.6	3.76	0.81	21.5	4.15	1.31	31.6
1922	1.78	0.52	29.2	2.02	1.07	53.0	4.30	4.52	105.1	2.48	3.85	155.2	5.94	3.27	55.1	5.03	1.39	27.6	4.36	1.06	24.3
1923	7.75	2.84	36.6	1.66	1.20	72.3	2.79	5.80	207.9	4.68	5.14	109.8	2.58	2.71	105.0	3.83	1.12	29.2	1.73	0.35	20.2
Mean	3.55*	2.28*	64.2	2.99*	1.94*	64.9	3.69*	4.88*	132.2	3.66	3.98	108.7	4.05	2.71	66.9	3.55	1.31	36.9	3.71	0.83	22.4

TABLE 1 — (*Continued*).

Year.	August.			September.			October.			November.			December.			For the Year.		
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1912	3.87	0.39	10.1	3.10	0.42	13.5	4.51	0.79	17.7	4.78	1.58	33.0	4.31	1.94	4.5	...	...	...
1913	4.91	0.32	6.5	4.84	0.65	13.4	10.82	4.06	37.6	3.78	2.74	72.4	2.74	2.12	77.4	50.24	25.74	51.3
1914	2.62	0.42	16.0	0.32	0.23	71.9	3.58	0.52	14.4	3.47	0.68	19.5	4.00	0.74	18.5	39.38	20.91	53.1
1915	8.63	2.65	30.7	3.50	1.14	32.6	3.08	1.35	43.8	3.43	1.39	40.5	5.30	2.65	50.0	49.05	24.25	49.5
1916	2.94	0.45	15.3	4.93	0.62	12.6	1.54	0.55	35.7	3.10	0.83	26.8	2.83	0.85	30.0	37.68	20.99	55.7
1917	5.12	0.80	15.6	0.67	0.32	47.8	8.35	2.87	34.4	1.10	1.73	157.3	3.17	1.27	40.1	44.15	25.13	56.9
1918	2.60	0.50	19.3	6.90	0.90	13.0	1.00	0.77	77.0	2.80	1.27	45.2	3.50	2.23	63.7	41.70	21.90	52.5
1919	3.82	0.56	14.7	5.49	1.18	21.5	2.79	0.88	31.5	4.91	2.38	48.4	2.22	2.24	100.1	47.31	26.41	55.8
1920	3.39	0.93	27.4	7.86	0.93	11.8	0.66	1.79	271.2	4.63	2.24	48.4	5.27	4.15	78.7	51.78	33.92	65.5
1921	2.54	0.47	18.5	3.96	0.33	8.3	1.50	0.44	29.3	5.08	1.16	22.8	2.14	1.45	67.7	40.35	21.29	52.7
1922	5.49	1.04	19.0	2.04	0.78	38.2	5.15	1.37	26.6	1.30	0.81	62.3	3.98	0.69	17.3	43.87	20.37	46.4
1923	2.46	0.30	12.2	3.86	0.40	10.4	6.44	1.42	22.0	2.68	1.14	42.5	4.61	2.94	63.8	45.07	25.36	56.3
Mean	4.03	0.74	18.4	3.96	0.66	16.7	4.12	1.40	34.0	3.42	1.49	43.6	3.67	1.94	52.9	44.60*	24.21*	54.3

Drainage Area: 26.8 sq. mi., 1912 to 1917; 22.8 sq. mi. in 1917; 23.9 sq. mi. since 1917.
Rainfall: For 1912, average of Barnes, Bakersville and Douglas; 1913, 1914 and 1917, average Barnes and Bakersville; 1915–1916, average Barnes, Bakersville and Lyman; 1918, see — 1918, average Bakersville-Nepaug Dam.

* 11 years only.

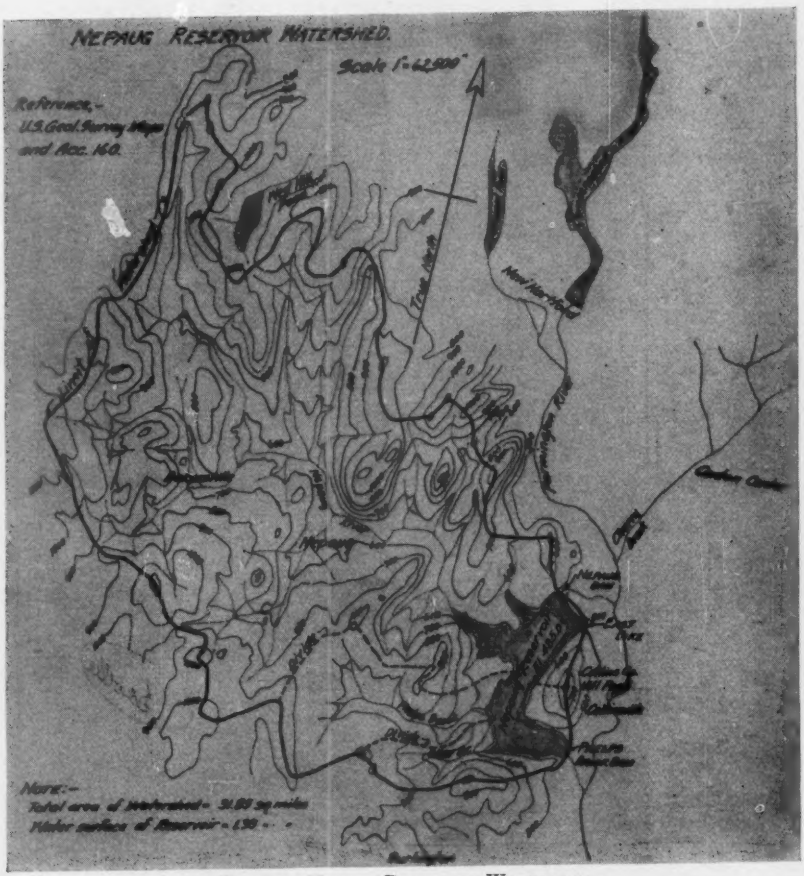


FIG. 1. — NEPAUG RESERVOIR WATERSHED.

Low annual rainfall is by no means a concomitant of local low run-off conditions. For instance, as stated above, during the summer and early fall of 1923 drought conditions were so severe as to put many water departments on short service, and in many localities the drought was said to have been nearly, if not quite, a record-breaker. Notwithstanding this, both the rainfall and run-off on the Hartford system for the calendar year were greater than those of the previous two years.

Year.	Rainfall, Inches.	Run-off.		Per Cent. Rainfall Collected.
		Inches.	c.f.s. sq. mi.	
1920	51.78	33.92	2.51	65.5
1921	40.35	21.29	1.57	52.7
1922	43.87	20.37	1.50	46.4
1923	45.07	25.36	1.87	56.3
Station average, 11 years	44.60	24.21	1.78	53.3

The Nepaug Basin of the Hartford water supply system is located about 16 miles northwest of Hartford (Conn.) on a branch of the Farmington River (Fig. 1). About 5 miles to the north of the Nepaug Reservoir is the so-called Compensating Reservoir, built by the Hartford Water Board to supply water in lieu of money to compensate mill owners on the Farmington River below Collinsville for damage done to their water powers by the diversion of the flow in the Nepaug River. The Compensating Reservoir is located on the East Branch of the Farmington River near New Hartford. The drainage area above the dam is about 62 sq. miles and in its northerly portion lies a portion of the southerly slope of the hills forming the western boundary of the Westfield-Little River watershed of the water-gathering grounds of the city of Springfield, Mass. To the north of the Nepaug Reservoir by less than 25 miles and on the westerly slope of the Connecticut River Valley, as also are the previously mentioned watersheds, lies the Manhan River watershed of the Holyoke (Mass.) water system.

All of these watersheds are in a thoroughly glaciated country, the country rock being generally gneiss or schist of the metamorphosed sedimentary rocks. Comparatively extensive areas of sand and gravel are scattered over all of these areas, and these are of great assistance both in conserving the run-off and improving the appearance of the water.

The Nepaug Reservoir has a tributary drainage basin of about 32 sq. miles. The drainage plan is divided as follows:

Clear Brook above weir	1.05 sq. mi.
Phelps Brook above weir	2.90 sq. mi.
Nepaug River above weir	23.90 sq. mi.
Area reservoir water surface	1.35 sq. mi.
Land area below weirs	2.65 sq. mi.
Total area	31.85 sq. mi.

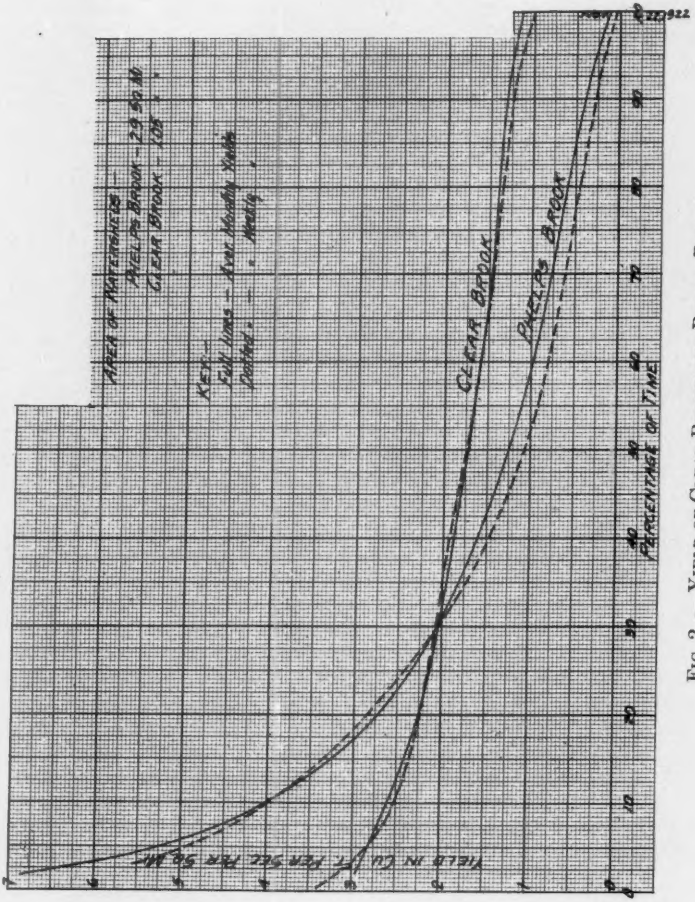


FIG. 2. — YIELD OF CLEAR BROOK AND PHELPS BROOK.

A very marked example of these conditions is found in two small contiguous drainage areas discharging into the Nepaug Reservoir (Fig. 2) —Clear Brook and Phelps Brook, with watersheds as given above. Clear Brook takes its rise in extensive deposits of gravel and sand, the water bubbling out from the bank over several acres and flowing rapidly to the reservoir. Not over a mile to the north Phelps Brook also discharges its waters. These are gathered for the most part from slightly rolling meadow country containing small swamps and little or no gravel deposits. The character of the run-off throughout the year is well shown in Fig. 2. The average color of the Clear Brook water is 15, with a maximum in 1923 of 21, while that of Phelps Brook is 36 with a maximum of 50. As to sustained flow in periods of drought: On Clear Brook the lowest run-off of record was 0.87 cu. ft. per sec. per sq. mile with a mean of 1.78 and a maximum of 4.11. On Phelps Brook the lowest recorded flow (weekly average) was 0.03 cu. ft. per sec. per sq. mile, with an average of 1.74 and maximum of 9.94. The per cent. of collection to rainfall on Clear Brook is 53.5 and on Phelps Brook 52.0% for the years of record.

Within the limits of the Nepaug watershed there are four standard Weather Bureau rain gages and one automatic Friez rain gage. There are standard concrete weirs with sharp crests and full contracted sides on the Nepaug River, and Phelps and Clear Brooks, all with automatic registers, and these stations are all kept under constant oversight. On the watershed of the East Branch of the Farmington River above the Compensating Reservoir dam there are four rainfall stations and a gaging station with an automatic Bristol gage which is checked against a nearby staff gage. It is from the records of the instruments on these watersheds that the following data have been compiled, and no effort has been spared to obtain reliable information.

The following tables are presented as giving minimum yields for various periods of time from the Nepaug basin since 1912 and to these, for purposes of comparison, other data have been added, through the courtesy of Messrs. W. E. Foss, E. E. Lochridge, and P. J. Lucey, engineers, respectively, of the Boston Metropolitan Water Works and those of Springfield and Holyoke, Mass.

Table 2 shows the average run-off for the driest consecutive months for the years of record. The original data from all of these, except the Nashua River, were for land surface only. In the case of the Nashua River (Boston Metropolitan Water Works) the data supplied were from the total watershed, including water surface of Wachusett Lake; this area was eliminated by deducting rainfall and evaporation* for the several months.

Table 3 shows the minimum yields for consecutive weeks on the contiguous watersheds of the Nepaug River and Phelps Brook, which are parts of the Nepaug Reservoir drainage system.

Figure 3 is presented as having in graphical form the low flow yield of

* Fitzgerald's determinations at Chestnut Hill Reservoir.

TABLE 2.

AVERAGE RUN-OFF FOR DRIEST CONSECUTIVE MONTHS FOR YEARS OF RECORD.

Years of Record — 1897–1922.			1908–1922.		1912–1922.		1897–1922.	
No. of Consecutive Months.	MANHAN RIVER.		LITTLE RIVER.		NEPAUG RIVER.		NASHUA RIVER.	
	Date.	M.G.D.	Date.	M.G.D.	Date.	M.G.D.	Date.	M.G.D.
1	Aug., 1913	0.07	Oct., 1910	0.02	Sept., 1914	0.13	Sept., 1914	0.156
2	July and Aug., 1913	0.09	July and Aug., 1913	0.04	July and Aug., 1913	0.17	July and Aug., 1913	0.170
3	July –Sept., 1913	0.13	Aug. –Oct., 1914	0.06	July –Sept., 1912	0.21	July –Sept., 1913	0.187
4	July –Oct., 1900	0.15	July –Oct., 1914	0.09	Aug. –Nov., 1914	0.265	July –Oct., 1910	0.221
5	July –Nov., 1914	0.18	June –Oct., 1914	0.11	July –Nov., 1914	0.29	July –Nov., 1910	0.236
6	July –Dec., 1914	0.20	June –Nov., 1914	0.18	July –Dec., 1914	0.31	July –Dec., 1910	0.261
7	June –Dec., 1914	0.21	June –Dec., 1914	0.19	June –Dec., 1914	0.335	July, 1910–Jan., 1911	0.331
8	July, 1910–Feb., 1911	0.26	July, 1910–Feb., 1911	0.27	June, 1921–Jan., 1922	0.46	July, 1910–Feb., 1911	0.362
9	June, 1910–Feb., 1911	0.36	July, 1910–Mar., 1911	0.37	June, 1921–Feb., 1922	0.48	June, 1910–Feb., 1911	0.426
10	June, 1910–Mar., 1911	0.45	June, 1910–Mar., 1911	0.46	May, 1921–Feb., 1922	0.56	May, 1910–Feb., 1911	0.459
11	July, 1910–May, 1911	0.52	May, 1910–Mar., 1911	0.47	Apr., 1921–Feb., 1922	0.72	Apr., 1910–Feb., 1911	0.518
12	July, 1910–June, 1911	0.54	Apr., 1910–Mar., 1911	0.57	July, 1914–June, 1915	0.835	Aug., 1910–July, 1911	0.573
15	July, 1910–Sept., 1911	0.51	Apr., 1910–June, 1911	0.75	June, 1914–Aug., 1915	0.84	July, 1910–Sept., 1911	0.489
18	July, 1910–Dec., 1911	0.69	Apr., 1910–Sept., 1911	0.69	July, 1921–Dec., 1922	0.81	May, 1910–Oct., 1911	0.540
24	June, 1909–May, 1911	0.85	Oct., 1909–Sept., 1911	0.91	Jan., 1921–Dec., 1922	0.99	Oct., 1909–Sept., 1911	0.741
30		...		...		...	Mar., 1909–Aug., 1911	0.840
36		...		...		...	Sept., 1908–Aug., 1911	0.808

TABLE 3.

MINIMUM YIELDS FOR CONSECUTIVE WEEKS, APRIL 1912 TO DECEMBER 1923, INCLUSIVE.

Actual "Mean Run-off" Figures per Square Mile.

PERIOD. Number of Lowest Consecutive Weeks.	NEPAUG RIVER. 1912–1923.			PHELPS BROOK. 1912–1923.		
	Run-Off.		Date.	Run-Off.		Date.
	M.G.D.	C.F.S.		M.G.D.	C.F.S.	
1	0.085	0.13	Sept. 23–29, 1917	0.02	0.03	Aug. 25–31, 1918
2	0.105	0.16	Sept. 16–29, 1917	0.03	0.05	Aug. 18–31, 1918
3	0.11	0.17	Sept. 13–Oct. 3, 1914 Aug. 3–23, 1914	0.05	0.08	Aug. 18–Sept. 7, 1918
4	0.11	0.17	Sept. 13–Oct. 10, 1914	0.05	0.08	Aug. 18–Sept. 14, 1918
5	0.115	0.178	Sept. 6–Oct. 10, 1914	0.079	0.122	Aug. 19–Sept. 22, 1923
6	0.126	0.195	July 13–Aug. 23, 1913	0.080	0.123	Aug. 12–Sept. 22, 1923
7	0.129	0.20	July 6–Aug. 23, 1913	0.082	0.127	Aug. 5–Sept. 22, 1923
8	0.136	0.21	June 29–Aug. 23, 1913	0.090	0.139	July 29–Sept. 22, 1923
9	0.146	0.227	June 22–Aug. 23, 1913	0.090	0.138	July 22–Sept. 22, 1923
10	0.156	0.241	July 15–Sept. 22, 1923	0.093	0.143	July 15–Sept. 22, 1923
11	0.158	0.245	July 8–Sept. 22, 1923	0.102	0.157	Aug. 5–Oct. 20, 1923
12	0.171	0.265	July 1–Sept. 22, 1923	0.106	0.163	July 29–Oct. 20, 1923
13	0.181	0.28	June 24–Sept. 22, 1923	0.104	0.16	July 22–Oct. 20, 1923
14	0.189	0.293	June 17–Sept. 22, 1923	0.105	0.162	July 15–Oct. 20, 1923
15	0.204	0.316	July 7–Oct. 19, 1912	0.114	0.176	July 8–Oct. 20, 1923
16	0.202	0.313	June 30–Oct. 19, 1912	0.144	0.224	July 1–Oct. 20, 1923
17	0.203	0.314	June 17–Oct. 13, 1923	0.174	0.269	June 24–Oct. 20, 1923
18	0.198	0.306	June 17–Oct. 20, 1923	0.195	0.30	June 17–Oct. 20, 1923
19	0.226	0.349	June 10–Oct. 20, 1923	0.235	0.364	June 10–Oct. 20, 1923
20	0.257	0.398	June 2–Oct. 19, 1912	0.250	0.386	June 30–Nov. 16, 1918
21	0.262	0.404	June 21–Nov. 14, 1914	0.253	0.391	June 23–Nov. 16, 1918
22	0.266	0.411	June 14–Nov. 14, 1914	0.259	0.40	June 16–Nov. 16, 1918
23	0.278	0.429	June 7–Nov. 14, 1914	0.265	0.41	June 9–Nov. 16, 1918
24	0.292	0.451	June 14–Nov. 28, 1914	0.273	0.421	June 2–Nov. 16, 1918
25	0.295	0.455	June 21–Dec. 12, 1914	0.293	0.453	May 26–Nov. 16, 1918
26	0.297	0.459	June 14–Dec. 12, 1914	0.306	0.473	June 9–Dec. 7, 1918

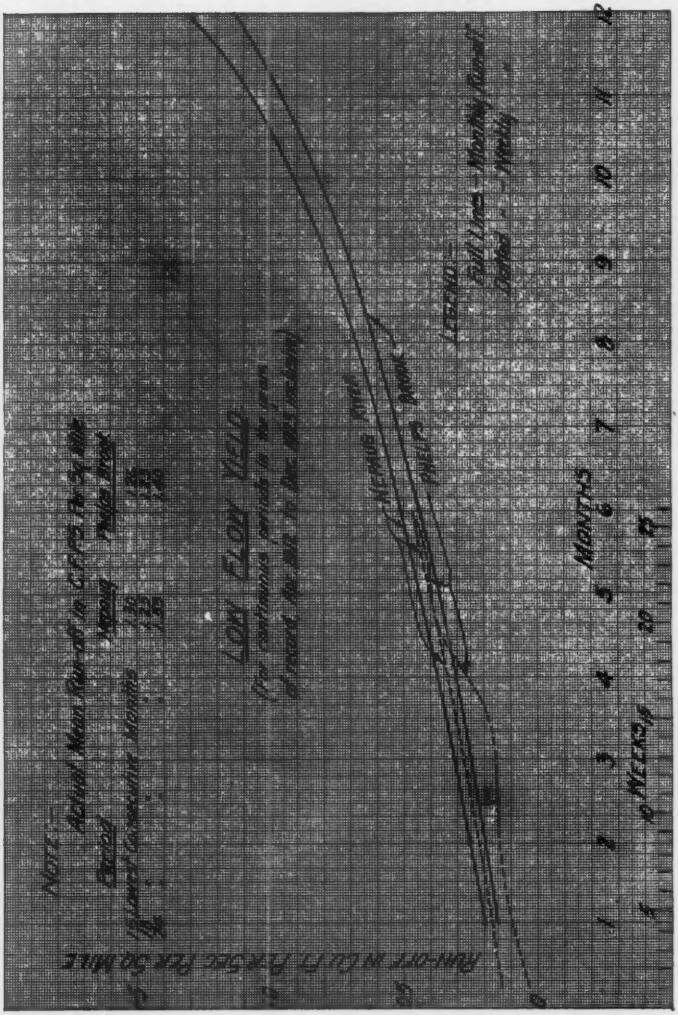


FIG. 3. — COMPARATIVE YIELD. PHELPS BROOK AND NEPAUG RIVER.

two contiguous streams and, incidentally, indicating by dotted lines the differences that may result from using monthly averages instead of those for shorter periods.

TABLE 4.

RAINFALL AND RUN-OFF, NEPAUG RESERVOIR.

Year.	RAINFALL. Nepaug Dam, Inches.	RUN-OFF. Nepaug Weir, Inches.	Per cent., using Nepaug Dam Rainfall.	Per cent., using Average Rainfall on Total Area.
1913	49.85	25.88	51.9	51.3
1914	40.97	20.97	51.2	53.1
1915	43.93	23.05	52.4	49.5
1916	41.01	20.83	51.1	55.7
1917	40.44	25.01	61.8	56.9
1918	36.56 min.	21.85	59.8	52.5
1919	46.43	26.56	57.2	55.8
1920	50.30 max.	33.90 max.	67.4	65.5
1921	40.01	21.24	53.1	52.7
1922	43.28	20.29 min.	46.9	46.4
1923	45.93	25.22	50.5	56.3
Totals	478.71	264.80	....	....
Average	43.52	24.07	55.2	54.3

This table is presented only for purpose of comparison of the rainfall and run-off in the several years and to show the difference which may occur if one rain gage record is used or a number from which representative conditions may be obtained. The rainfall data are from the automatic gage at the Nepaug dam which is located at the bottom of the watershed. The best probable exhibit of actual conditions is found in Table 1, where the rainfall given is the mean of several stations located in the watershed area, for the purpose of getting as nearly as possible the average amount of precipitation on the whole area.

	Inches Rainfall.	Inches Run-off.	Ratio Run-off to Rainfall.
Storage period (delivery period)	21.49	17.61	82.1%
Growing period	11.69	2.94	25.2%
Replenishing period	11.42	11.42	3.1%
Total	44.60	24.21	54.3%

Dividing the years of record into the periods suggested by the late Mr. George W. Rafter,* the above tabulation presents interesting conditions as to the disposal of precipitation on this watershed. Mr. Rafter gave the following reasons for his divisions which, he is careful to state,

* Water Supply and Irrigation Paper No. 80, U. S. Geological Survey, "The Relation of Rainfall to Run-off."

are more or less arbitrary: the Storage period, December to May inclusive, when evaporation and absorption by plants are relatively slight, and a relatively large proportion of the precipitation appears as stream flow; the Growing period, June to August inclusive, the period of active vegetation when evaporation and absorption by plants are at a maximum; the Replenishing period, September to November inclusive, when with normal rainfall the ground water reservoir tends to refill.

A study of the records of rainfall and run-off for the years of record on the Nepaug watershed (1912 to 1924, Table 1) shows that although 1923 (Fig. 7) was not the minimum yield year, during 19 summer weeks — June 10 to October 20 — it produced the lowest yield for the 11 years of record on the Nepaug watershed, *i.e.*, about 0.35 cu. ft. per sec. per sq. mile for a period of over 4 months. (See Table 3.) With the exception of the 15 and 16 week periods, the year 1923 produced the lowest yields on the Nepaug watershed (1912–1924) for consecutive weekly periods of low flow from the tenth to the twentieth, averaging for these periods flows from about $\frac{1}{4}$ to $\frac{1}{3}$ million gal. per sq. mile per day (land surface). This is of especial interest when it is recalled that the water loss by evaporation alone during this period averages not far from ten times this amount per sq. mile of water surface.

That low flow conditions of this year were not of local importance only, is attested by a statement made by Mr. R. A. Hale,* chief engineer of the Essex Company, Lawrence, Mass.: " We had four months of quite low yield (Merrimac River), some small rivers of 70 sq. miles drainage area yielding 0.10 cu. ft. per sec. per sq. mile, which shows the effect of the ground water in our large watersheds when the minimum yield is 0.25 cu. ft. per sec. per sq. mile."

The four diagrams of accumulated rainfall and run-off for the Nepaug watershed for the years 1920, 1921, 1922, and 1923 (Figs. 4, 5, 6, and 7) are given for purposes of comparison. That for 1921 is for a year of very low rainfall. The accumulated rainfall was below the station average during the entire period and at the end of the year had a deficiency of 6.69 in. On the other hand, there were no serious drought conditions, due to the fact that the rainfall, while low, was distributed quite evenly throughout the year. For the year 1922 the total run-off was a minimum for the period, being 3.78 in. below station average, while the rainfall itself was nearly normal. The low run-off condition was due, first, to depleted ground storage at the end of the previous year, and to low precipitation during the spring and early summer months, which did not allow the ground storage to fill completely before the growing period for vegetation came on with its demand for water. The total rainfall was helped out by a storm of considerable magnitude in the early part of September which, however, was not sufficient to fully restore the depleted ground storage. The drought of the year 1923, like that of 1922, was due in a measure to de-

* Personal letter.

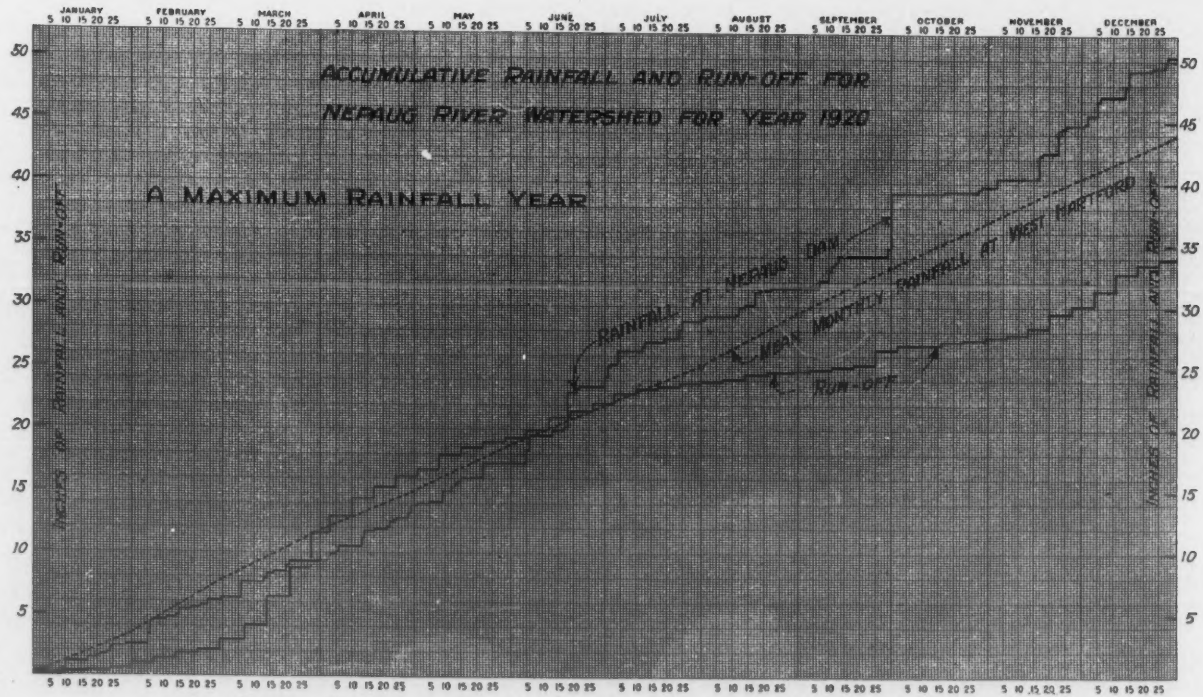


FIG. 4. — RAINFALL AND RUN-OFF FOR NEPAUG RIVER WATERSHED FOR YEAR 1920.

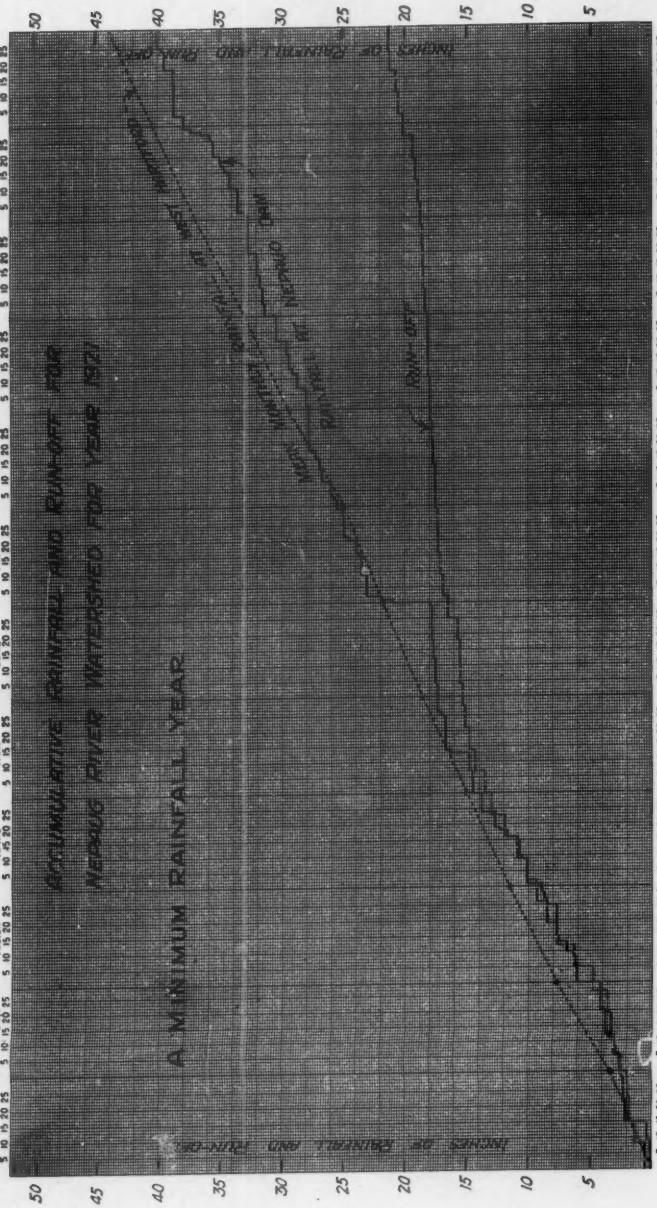


FIG. 5. — RAINFALL AND RUN-OFF FOR NEPAUG RIVER WATERSHED FOR YEAR 1921.

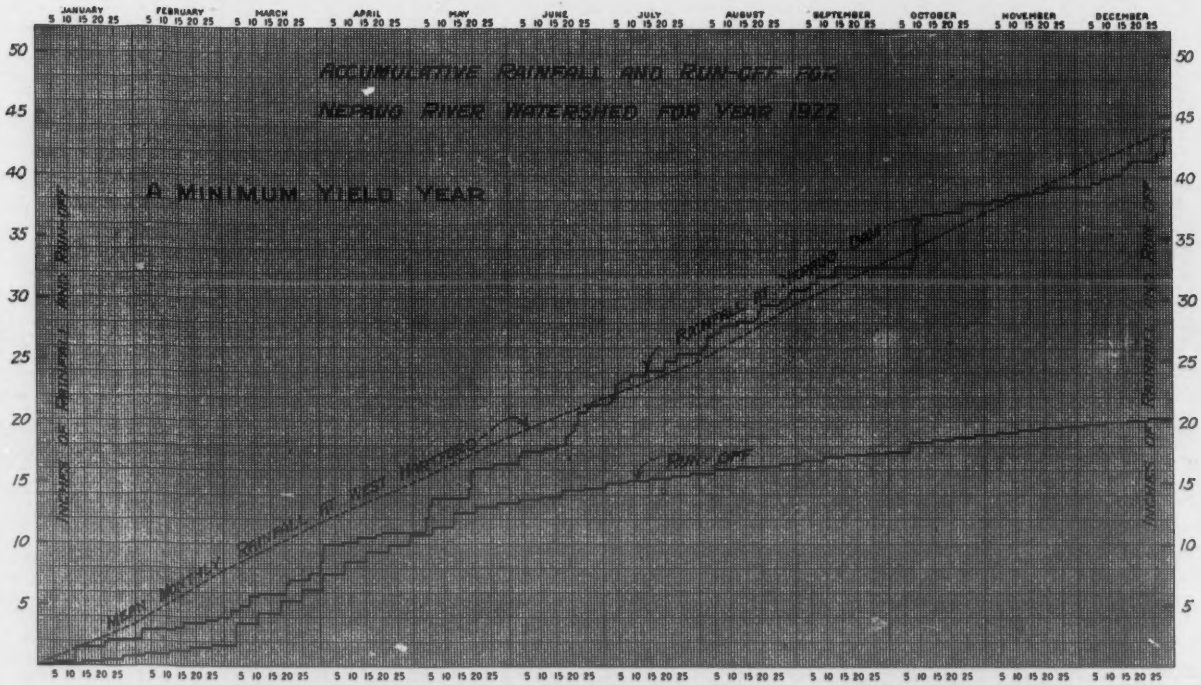


FIG. 6. — RAINFALL AND RUN-OFF FOR NEPAUG RIVER WATERSHED FOR YEAR 1922.

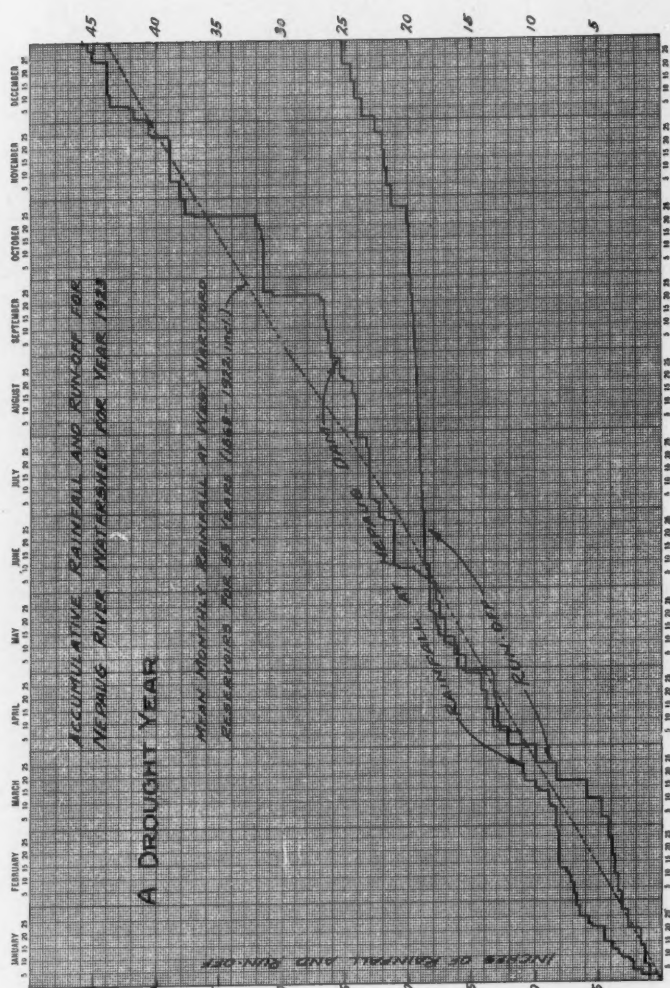


FIG. 7. — RAINFALL AND RUN-OFF FOR NEPAUG RIVER WATERSHED FOR YEAR 1923.

pleted ground storage of the previous year, which was somewhat mitigated by the high run-off in March, April, and May.

Using Birnie's* deductions, the probable extreme deviation of the mean from the 11 years of record of the Nepaug basin, as used here, would be about 8%. It is likely that it is much closer than that because of the extremes that have occurred during the period. The outstanding conditions to be considered in preparing for drought periods during the summer months are condition of the ground storage at the end of the calendar year and precipitation during the early months of the following year; also, continuous precipitation, even if slight, through the summer is much more conducive to normal run-off than are heavy storms after a period of drought. This last condition is well shown in the Accumulated Rainfall and Run-off diagram for 1923 (Fig. 7).

Although there was a rainfall of 4.54 in., September 23–24, after over three months' drought, yet hardly a ripple is noticed in the run-off curve; and after the 5.62 in. downpour of October 24–25 only 1.11 in., or less than one-fifth, appeared as run-off. In water supply practice it is the yield rather than the amount of precipitation that is of primary importance, and this depends upon many diverse factors — as slope and shape of the watershed, character of the rock cover, and opportunity for ground storage, and others of more or less importance as the seasons change.

At the present time, on account of so many diverse relationships that have not been correlated either in observation or theory, it is impossible to deduce even approximately accurate forecasts for floods or droughts. On the other hand, such intensive study is being given to these conditions by eminently capable investigators, from a theoretical standpoint, and such advance is being made in observation and reliable record of actual facts, that it seems almost certain that some of the present problems will be found capable of solution, or at least of reliable approximation.

Flood phenomena, although usually spectacular, are much more easily recorded than those of drought, which creep on almost without warning. For this reason more reliable data are available for establishing precautionary measures properly to meet such conditions. It is now proposed to give some description of the great floods of record in this district, and present data concerning lesser flood conditions that have been carefully observed in the Hartford water supply system. Although the greater floods are those that do damage to life and property, they come so suddenly and go so quickly that means of observation are usually lacking regarding details. Study of the lesser flood conditions, therefore, may show certain relative principles that will be applicable to the problems of those of greater magnitude.

Floods due to storm run-off from New England streams have two principal types: those due to heavy and continuous precipitation during the

* "On Mean or Average Rainfall and the Fluctuations to Which It Is Subject." Alex. R. Birnie, Proc. Inst. C. E., Vol. cix (1892), pp. 89–172.

warm months, and those due to occurrence of heavy rainfall in conjunction with a thaw in the winter and spring at a time when the surface of the ground is usually sealed with ice, and may be covered with snow.

The storm and flood of October 3 and 4, 1869, was typical of the first kind; those on the Farmington River and neighboring streams in April, 1924, and that on Stony Brook (Boston) in February, 1886, are representative of the second kind.

The flood of October 2–5, 1869, was apparently a record-breaker, not only because of the quantity of rain which fell but also from the wide area over which the storm extended.

The amount of rainfall in this storm and its rate, producing the flood, was unprecedented in records; nor has it been equalled since. The storm center was near Canton, Conn., about 13 miles northwest of Hartford, and about 2 miles east of Nepaug Reservoir. This storm was quite fully described by James B. Francis, M. Am. Soc. C. E.* The distribution of rainfall is shown on the isopluvial chart (Plate I).†

Over 150 stations reported this storm, extending from Virginia to Canada and west to about the middle of Pennsylvania. It was therefore a coast storm with two foci, the maximum in Connecticut with 12.35 in. and the second in eastern Pennsylvania with about 8 in.

Little definite information is of record regarding the storm run-off. A. J. Henry‡ records it as "probably the greatest of all 'rain floods' in New England." In point of magnitude, on the Connecticut River gage at Hartford the flood produced by this storm ranks sixth in a period of 75 years. In commenting on this storm, Mr. Henry remarks: "The lesson to be drawn from the foregoing appears to be, that although summer floods — or, more strictly speaking, what may be called 'rain floods' — occur very infrequently in New England, the probability of such a flood cannot be disregarded. The occurrence of flood-producing rains in the warm season, such as those of October, 1869 and 1897, seems to be due to an extraordinary combination of atmospheric conditions, such as may occur but once in a lifetime."

On December 10, 1878, occurred a storm with a run-off that produced a 25.2 gage height on the Connecticut River, which was 0.7 ft. higher than that of October, 1869. While no authentic information is at hand concerning conditions producing this flood, it is very probable that there was an unusual combination of at least considerable depth of snow on the ground, together with a warm rain. At any rate, this flood is reported to have been the greatest on the Farmington River above Hartford, carrying away many bridges and threatening several of the power dams on the river. A height of 10.5 ft. over the dam is reported at Greenwood's Mills at New Hartford, and a study of conditions leads to the conclusion that this flood

* Transactions, Am. Soc. C. E., Vol. vii, Aug., 1878, p. 224 *et seq.*

† Reproduced through courtesy of the Am. Soc. C. E.

‡ "Floods on New England Rivers," *Monthly Weather Review*, Dec., 1914, pp. 682–686.

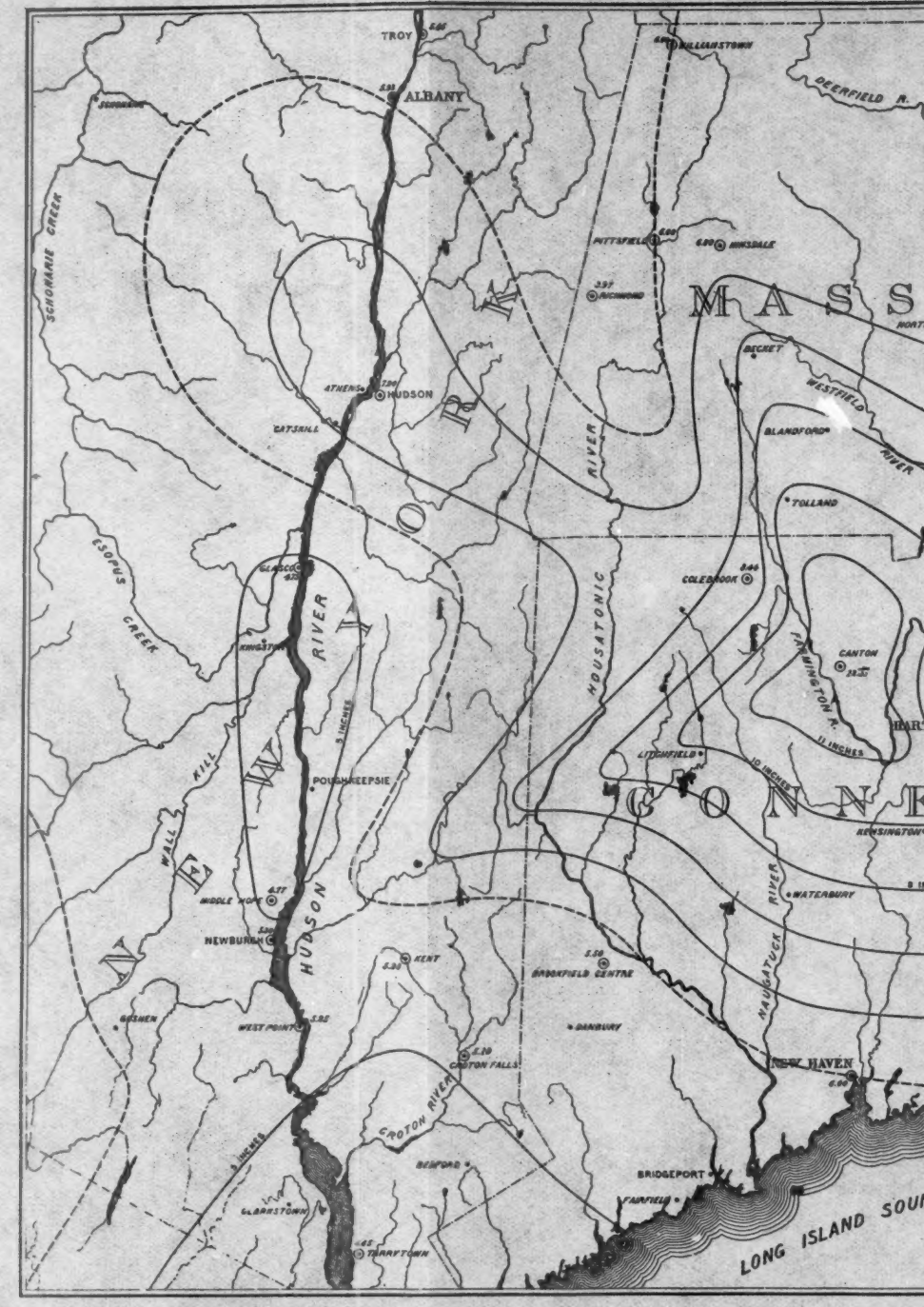
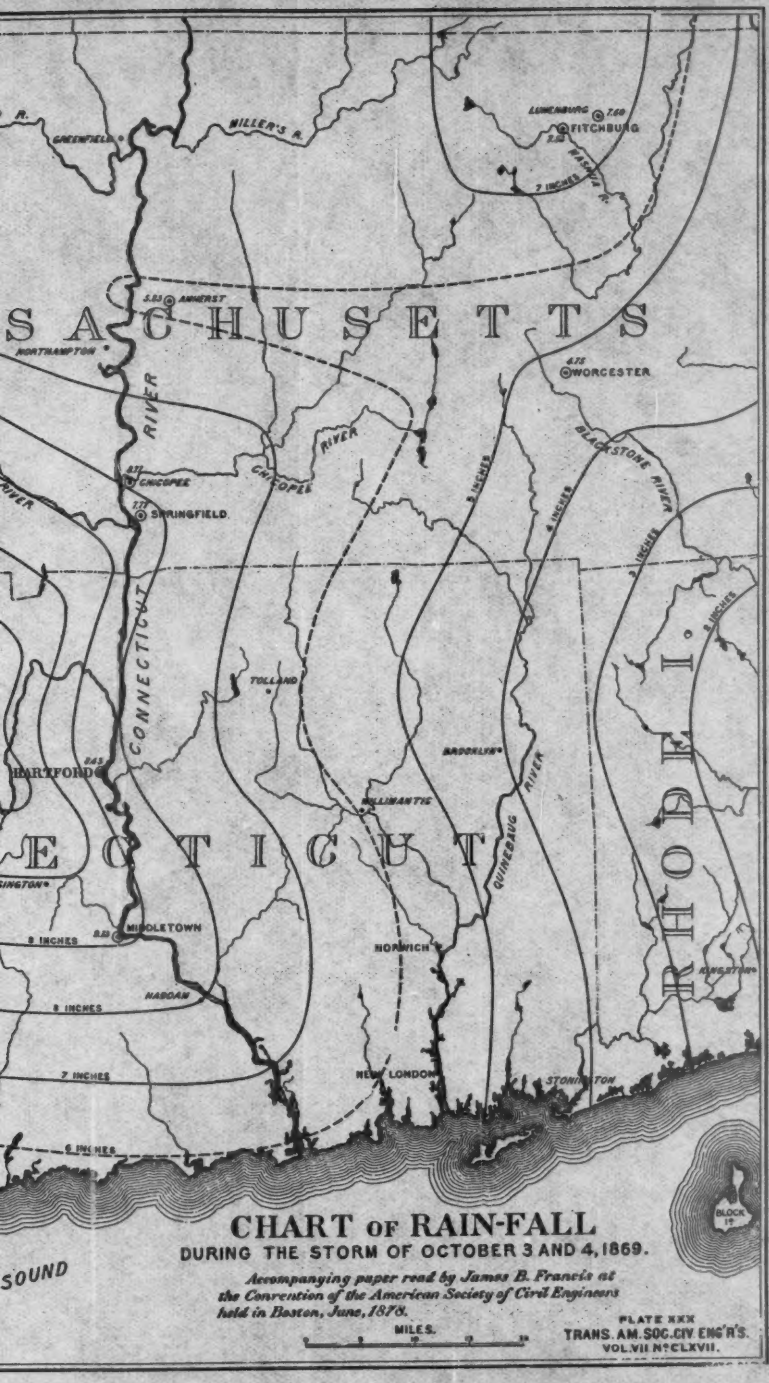
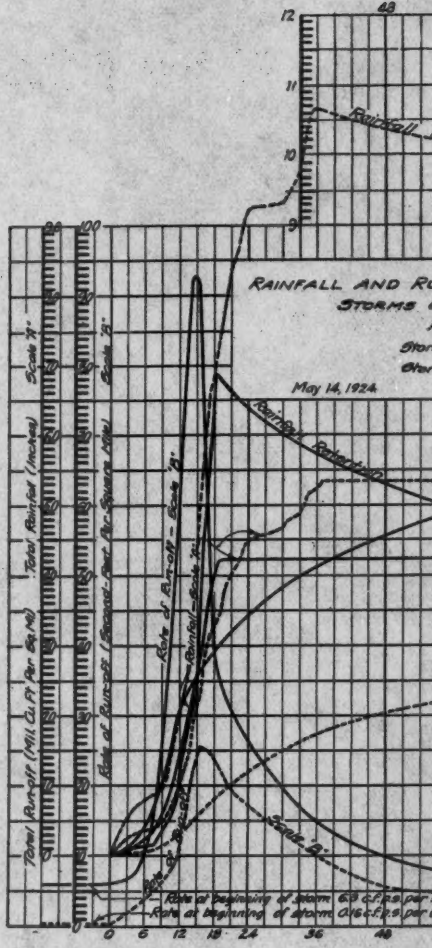

TROY
ALBANY
SCHOHARIE CREEK
ATHENS
HUDSON
CATSKILL
ESOPUS CREEK
KINGSTON
POUGHKEEPSIE
WALL KILL
MIDDLE HOPE
NEWBURGH
HUDSON
WEST POINT
CROTON RIVER
CROTON FALLS
KENT
BRIDGEFIELD CENTRE
DANBURY
NEW HAVEN
WATERBURY
NAUGATUCK RIVER
BRIDGEPORT
FAIRFIELD
LONG ISLAND SOUND
HOUSATONIC RIVER
COLEBROOK
LITCHFIELD
FARMINGTON R.
CANTON
NEWINGTON
BECKET
WESTFIELD
BLANDFORD
TOLLAND
PITTSFIELD
HINSDALE
RICHMOND
DEERFIELD R.
WILLIAMSTOWN
GLENS FALLS
TARRYTOWN
BEDFORD
GLARRISTOWN
5 INCHES
10 INCHES
11 INCHES
NEW YORK
MASS
CONN

PLATE I.
N. E. W. W. ASSOCIATION.
VOL. XXXIX.
SAVILLE ON
SOME FLOODS AND DROUGHTS
OF NEW ENGLAND STREAMS.





RAINFALL AND RUN-OFF ON NEPA
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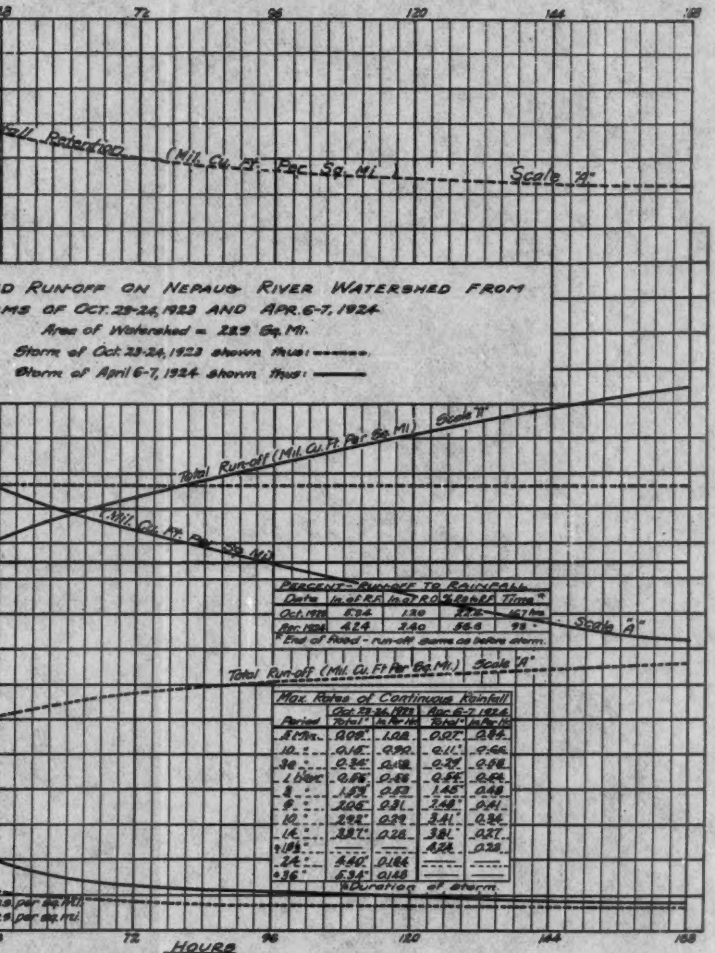

72
96
120
144
168
Scale ½"
(Mil. Cu. Ft. Per Sq. Mi.)
D RUN-OFF ON NEPAUG RIVER WATERSHED FROM
MS OF OCT. 23-24, 1923 AND APR. 6-7, 1924.
Area of Watershed = 28.9 Sq. Mi.
Storm of Oct. 23-24, 1923 shown thus: -------
Storm of April 6-7, 1924 shown thus: ———
Scale 1"
Total Run-off (Mil. Cu. Ft. Per Sq. Mi.)
Scale ½"
Scale "A"
Total Run-off (Mil. Cu. Ft. Per Sq. Mi.)
Scale "A"
HOURS

PERCENT - RUNOFF TO RAINFALL				
Date	In. of RA	In. of RO	% Equiv.	Time*
Oct. 1923	2.24	1.30	57.8	42 hrs
Apr. 1924	4.24	3.40	80.2	92 "

* End of flood - run-off same as before storm.

Max. Rates of Continuous Rainfall				
	Oct. 23-24, 1923		Apr. 6-7, 1924	
Period	Total In.	In. Per Hr.	Total In.	In. Per Hr.
5 Min.	0.09"	1.08	0.07"	0.84
10 "	0.16"	0.90	0.11"	0.66
30 "	0.34"	0.68	0.21"	0.42
1 Hour	0.56"	0.56	0.35"	0.35
3 "	1.53"	0.51	1.45"	0.48
9 "	2.06"	0.31	2.68"	0.41
10 "	2.92"	0.29	3.41"	0.34
14 "	3.27"	0.23	3.81"	0.27
*18 "	———	———	4.24	0.23
24 "	3.40"	0.184	———	———
*36 "	3.94"	0.146	———	———

*Duration of storm.

EPAUG RIVER WATERSHED FROM STORMS OCTOBER 23–24, 1923,
AND APRIL 6–7, 1924.

may have produced a run-off at its peak of perhaps 16 000 cu. ft. per sec., or 69.2 cu. ft. per sec. per sq. mile of drainage area.

On March 1, 1896, — according to Collins Company records at Collinsville on the Farmington River, — there was a flood almost as great as that of 1878. This flood was caused by a hard rain falling on about a foot of snow.

A noteworthy flood of recent years on the Farmington River was on February 13, 1900, and computations from conditions at Greenwood's dam indicate a maximum run-off of about 11 500 second ft., or about 50 cu. ft. per sec., per sq. mile.

Leaving these historical floods, which are chiefly remembered because of the damage caused by them, it is now proposed to take up the details of several excessive run-off periods in this same locality, concerning which reliable data are at hand. It is not proposed to treat them in chronological order, as there seems to be no particular reason for so doing.

In point of magnitude, the flood caused by the storm of April 7, 1924 (Plate II), is by far the most important of recent years and, possibly, was the greatest of this century on the East Branch of Farmington River. In comparing it with earlier floods, its height at Collinsville was 20 in. below the mark recording high water of the 1878 flood, but whether or not this is due to changes in the overflow works, or other local conditions, it is impossible to state now.

In the storm of April, 1924, the rain fell for about 19 hours almost without intermission, and at the end of this period about 4.25 in. had been recorded. The ground was frozen, and here and there were patches of snow, but there was no larger amount of run-off from this cause on the Hartford watersheds. Further north, the contrary was the case, P. J. Lucey, engineer of the Holyoke Water Board, stating that the rainfall on the Manhan drainage area was 3.49 in. from the beginning of the storm to the crest of flood, and the ground was covered with from 6 to 8 in. of soft snow. Weather Bureau reports from Amherst, Springfield, Turners Falls, and Worcester (Mass.), as well as from Bridgeport, Colchester, New Haven, Torrington, and Waterbury (Conn.), all reported no snow on the ground. None of the Weather Bureau stations, either in the vicinity of the Hartford or the Holyoke watersheds, recorded as much rain as these stations; which leads to the conclusion that the storm was of local intensity in the high lands bordering the westerly side of the Connecticut River Valley. Several highway bridges were washed out, and the roadbed of the railroad between Collinsville and New Hartford, Conn., was washed from under the tracks. As to the quantity of water discharged, the records at the peak of the flood indicate rates in cubic ft. per second per sq. mile of 52 for the west branch of the Farmington River (slightly less than flood of 1878) at New Hartford, 231 sq. miles; 73 from the east branch at Richards Corner Dam, 61.2 sq. miles; and 90 from the Nepaug River at the upper weir, 31.85 sq. miles. For the same flood the Holyoke Water Department* had 210 cu. ft. per sec.

* P. J. Lucey, Engineer.

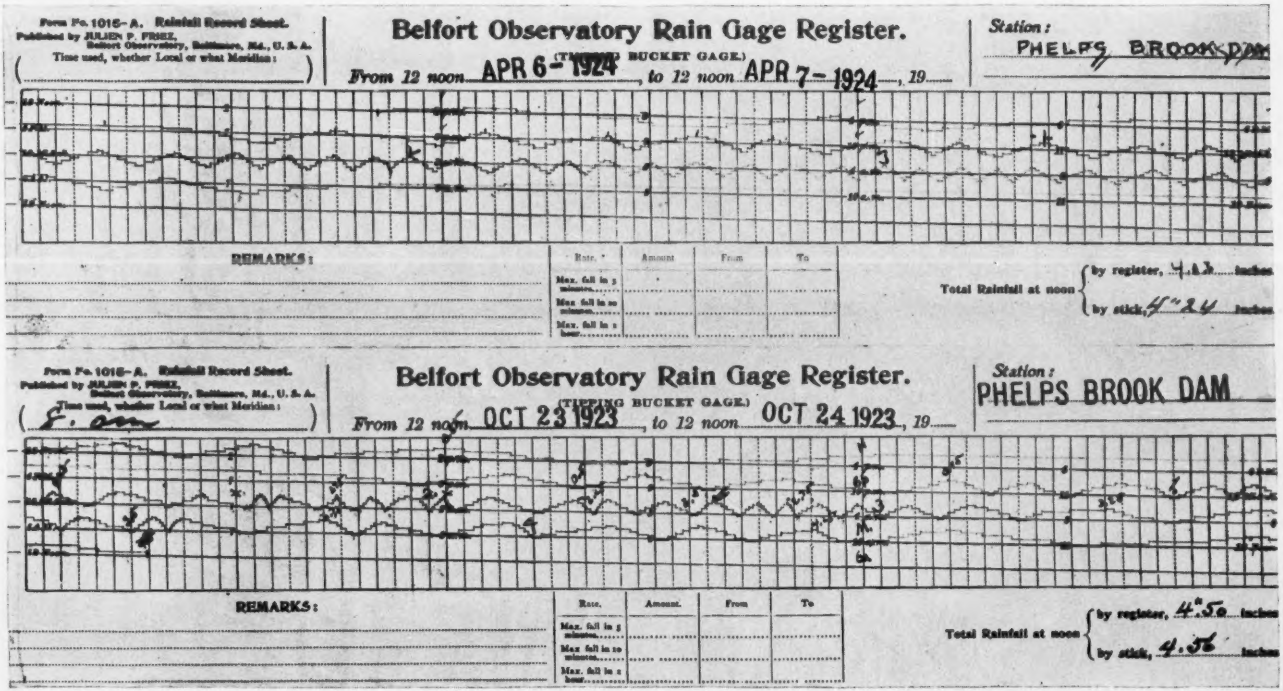

Form No. 1016–A. Rainfall Record Sheet.
Published by JULIEN P. FRIEZ,
Belfort Observatory, Baltimore, Md., U. S. A.
Time used, whether Local or what Meridian:
Belfort Observatory Rain Gage Register.
(TIPPING BUCKET GAGE.)
From 12 noon APR 6 – 1924, to 12 noon APR 7 – 1924, 19
Station: PHELPS BROOK DAM
REMARKS:
Rate.
Amount.
From
To
Max. fall in 5 minutes
Max fall in 10 minutes
Max. fall in 1 hour
Total Rainfall at noon
by register, 4.53 inches
by stick, 4" 24 inches
Form No. 1016–A. Rainfall Record Sheet.
Published by JULIEN P. FRIEZ,
Belfort Observatory, Baltimore, Md., U. S. A.
Time used, whether Local or what Meridian:
E. Dan
Belfort Observatory Rain Gage Register.
(TIPPING BUCKET GAGE.)
From 12 noon OCT 23 1923, to 12 noon OCT 24 1923, 19
Station: PHELPS BROOK DAM
REMARKS:
Rate.
Amount.
From
To
Max. fall in 5 minutes
Max fall in 10 minutes
Max. fall in 1 hour
Total Rainfall at noon
by register, 4.50 inches
by stick, 4.56 inches

FIG. 8. — RAINFALL RECORDS. STORMS OF OCTOBER 23–24, 1923, AND APRIL 6–7, 1924.

per sq. mile from the Tucker Brook watershed of 5.75 sq. miles, and a probable run-off from the 13 sq. miles above the reservoir of about 203 cu. ft., allowing for reservoir storage. Mr. Lucey states: "Our previous maximum flood occurred February 13, 1900, when the run-off was 183 cu. ft. per second, per sq. mile, the covering of the ground being generally icy and favorable to a quick run-off." The rainfall producing this latter run-off was 2.37 in. before the peak of run-off, with a total of about 3.55. The rainfall producing the peak of the flood in the Nepaug River, mentioned above, was about 2.6 in. for the flood of October, 1923.

Comparing the rainfall and run-off on the Nepaug and Manhan areas, it appears that the rainfall up to the crest of the flood was nearly an inch less at the Holyoke station than at Nepaug, while the discharge at the latter place was only about one-half that at Holyoke. As the difference in rainfall would hardly account for the great difference in run-off, it would appear probable that the cause might be found in the slopes and shapes of the two watersheds as well as the accumulated snow on the ground. The Nepaug watershed is roughly circular or oval, while that of the Manhan River is long and narrow.

The flood, or, rather, excess run-off in the storm of October 23–24, 1923 (Plate II), was due solely to heavy rainfall. That it was not of great magnitude was due to the condition of the ground and height of ground water table. The rainfall was about $5\frac{1}{3}$ in., but less than a quarter of it appeared in the run-off. The storm came after a long period of severe drought, which has previously been mentioned, and is well shown on the diagram of accumulated rainfall and run-off for 1923. The rainfall detail of these two storms is shown on Fig. 8.

Due to the condition of the ground, and the avidity with which water was absorbed, only from 15 to 25% of the precipitation appeared as immediate run-off after a period of 167 hours; while in the case of the April,

TABLE 5.

NEPAUG RIVER WATERSHED — MAXIMUM FLOOD FLOWS — DURATION.

Rate for.	CU. FT. PER SECOND PER SQ. MILE.		
	Oct. 27, 1913.	Oct. 24, 1923.	Apr. 7, 1924.
3 hrs.	51.5	25.4	83.2
6 hrs.	50.9	24.0	70.3
12 hrs.	48.1	21.0	53.6
15 hrs.	46.3	19.9	47.7
18 hrs.	43.8	19.0	44.8
24 hrs.	39.6	17.2	37.7
36 hrs.	32.3*	14.4	29.9
48 hrs.	29.6	12.1	24.7

* Rate for 35 hours.

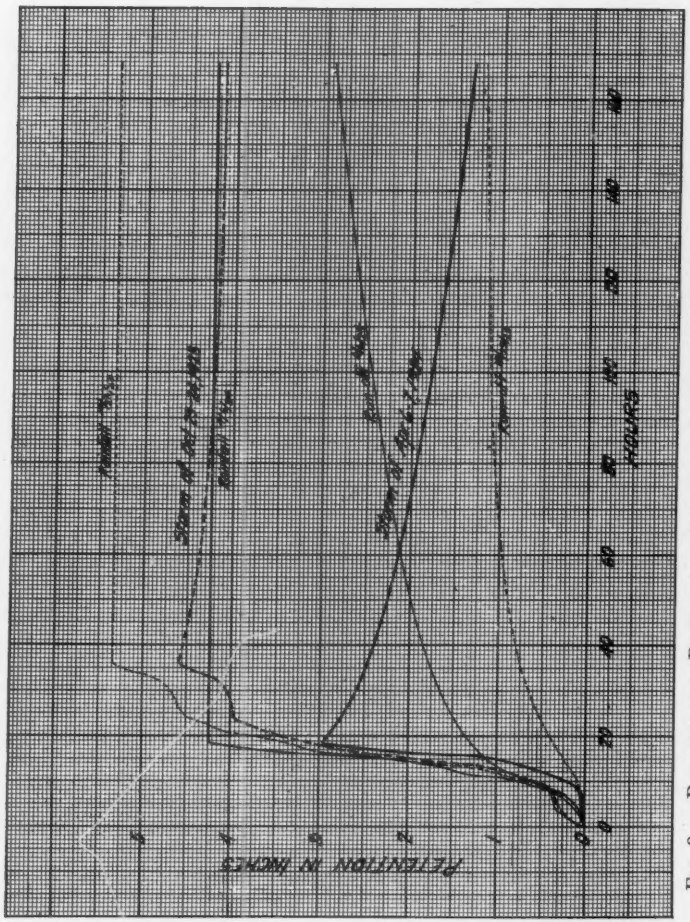


FIG. 9. — RAINFALL AND RETENTION. STORMS OF OCTOBER 23–24, 1923, AND APRIL 6–7, 1924.
NOTE: — Lines marked "Storm" indicate Retention of Rainfall.

1924, storm, nearly 55% appeared within 93 hours of the beginning of the storm and about 70% after 167 hours (Fig. 9). The rates and period of run-off are shown in Table 5 and on Fig. 10.

The heaviest rainfall that has been recorded in the 11 years of record on the Nepaug watershed was that of October 24–26, 1913, when an average of about 6.73 in. fell in about 36 hours, 6.7 in. falling between 4.30 P.M. of October 25 and 6 P.M. of October 26. This rainfall produced a flood height of about 52 cu. ft. per sec. per sq. mile.

Another excessive run-off for this locality was recorded by Mr. W. L. Butcher,* in *Engineering News*, January 4, 1912, p. 37, when on a small drainage area of about 1.72 sq. miles, in October of the previous year a sudden rain storm of about 4.75 in. produced a flood peak of about 62 cu. ft. per sec. per sq. mile, and an average rate of 45 cu. ft. per sec. for 30 hours (vicinity of New Hartford, Conn.).

From what has been observed on the watersheds of Hartford and vicinity it appears that run-offs of from 50 to 60 cu. ft. per sec. per sq. mile are of frequent occurrence; that flood discharge of 90 to 100 cu. ft. per sec. per sq. mile may be expected occasionally, say once in 25 or 30 years; and those of 200 cu. ft. per sec. or thereabouts are to be looked for at rare intervals when combinations of ground conditions, stored snowfall, rising temperature, and circumstances producing heavy precipitation occur. A comparison of various flood discharges is shown on Fig. 11.

That stored snowfall and rising temperature are principal factors in the high-discharge spring freshets everyone knows, but it may be interesting to observe just what part this locked-up water may play. To this end there are presented a diagram and run-off figures of the discharge from the Nepaug watershed under conditions of rather high temperature without heavy rainfall.

From March 20 to April 8, 1916, the total precipitation amounted to about 0.59 in., falling on four different and widely separated days.

Snow measurements were made in a number of places, representative of conditions over the different Hartford watersheds, on two different dates (1916), with the following average results as to water equivalent:

March 20 — 16 in. snow depth, equivalent in water to	4.50 in.
March 28 — 9 in. snow depth, equivalent in water to	3.30 in.
Difference 7 in. snow depth, equivalent in water to	1.20 in.
Precipitation	.39 in.
Total run-off March 20–28	1.59 in.

From March 24 to April 8 there was only 0.2 in. precipitation, yet for the period March 27 to April 8 the average daily run-off from the retained water on 26.8 sq. miles was about 7 cu. ft. sec. per sq. mile, with four days well over 12.

* 245 State Street, Boston.

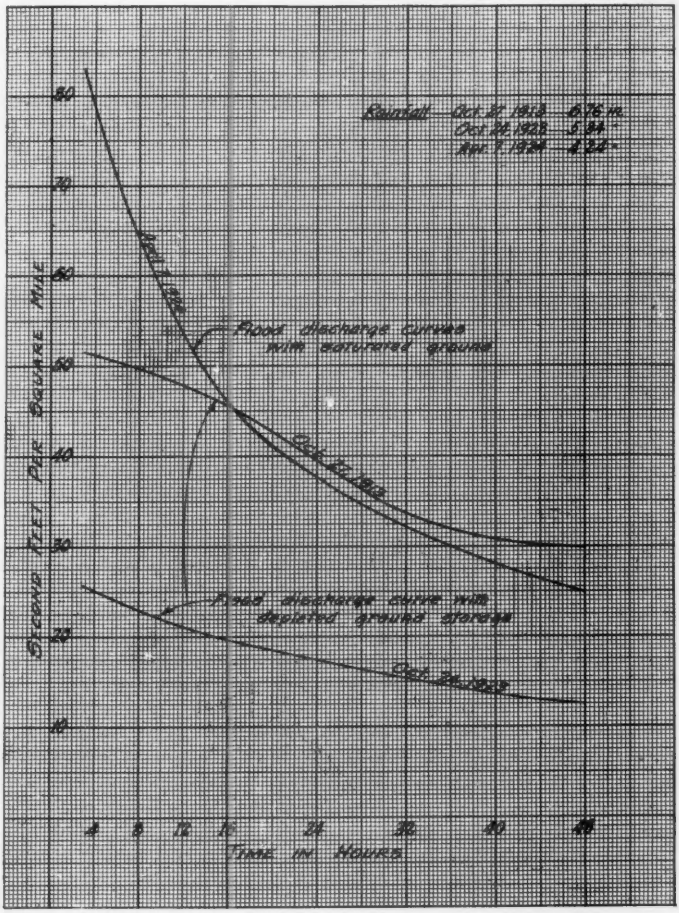

Rainfall — Oct. 2, 1913 — 6.76 in.
Oct. 29, 1927 — 5.84 "
Apr. 7, 1936 — 4.22 "
Flood discharge curve with saturated ground
Flood discharge curve with depleted ground storage
Oct. 29, 1927
Oct. 2, 1913
Ground level flow per square mile
Time in Hours

FIG. 10. — DURATION OF MAXIMUM FLOOD DISCHARGE.

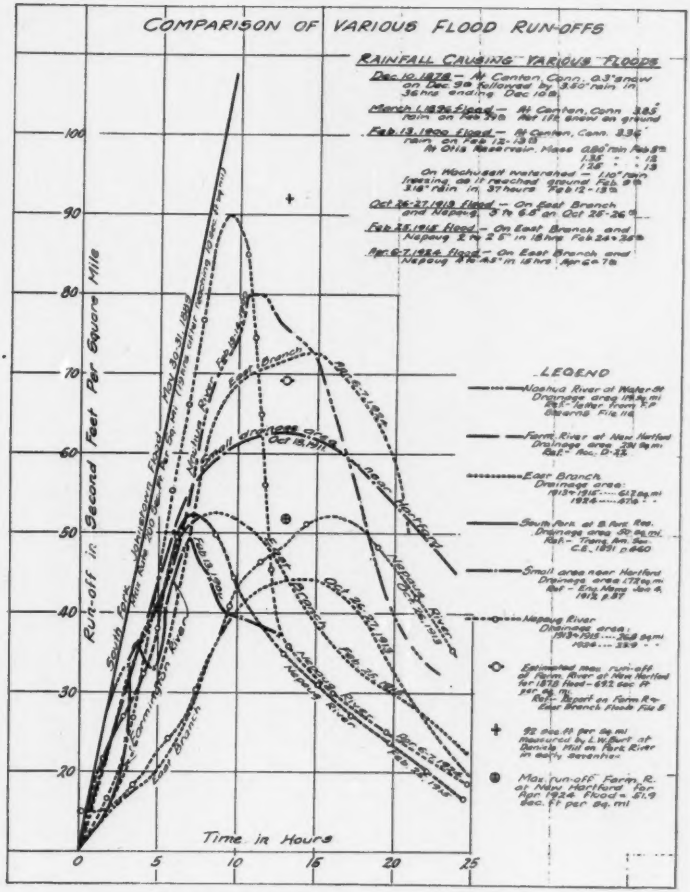

COMPARISON OF VARIOUS FLOOD RUN-OFFS
RAINFALL CAUSING VARIOUS FLOODS
Dec 10, 1878 — At Canton, Conn. 0.3" snow on Dec 9th followed by 3.30" rain in 36 hrs ending Dec 10th
March 1, 1936 Flood — At Canton, Conn 3.85" rain on Feb 29th. Had 11" snow on ground
Feb 13, 1900 Flood — At Canton, Conn. 3.36" rain on Feb 12-13th
At Otis Reservoir, Mass 0.80" rain Feb 8th 1.35 " " 12 1.26 " " 13
On Housatonic watershed — 1.10" rain freezing as it reached ground Feb 9th 3.18" rain in 37 hours Feb 12-13th
Oct 26-27, 1913 Flood — On East Branch and Nepaug 5" to 6.5" on Oct 25-26th
Feb 25, 1918 Flood — On East Branch and Nepaug 2.14" in 2.5" in 18 hrs Feb 24 + 25th
Apr 6-7, 1936 Flood — On East Branch and Nepaug 2.10-2.45" in 15 hrs Apr 6-7th
LEGEND
Naugatuck River at Waterbury Drainage area 178 sq mi Ref - letter from F.P. Stearns File 11a
Farm River at New Hartford Drainage area 29.1 sq mi Ref - Rec. D-42
East Branch Drainage area 1913 - 31st ---- 61.7 sq mi 1936 ---- 67.9 " "
South Park at S. Bur. Res. Drainage area 20.9 sq mi Ref - Trans. Am. Soc. C.E. 1891 p 640
Small area near Hartford Drainage area 1.73 sq mi Ref - Eng. News Jan 4, 1912 p 27
Nepaug River Drainage area 1913-1915 ---- 248 sq mi 1936 ---- 339 " "
Estimated max. run-off at Farm River at New Hartford for 1878 flood - 6.92 cfs per sq mi Ref - Report on Farm River East Branch Floods File 8
92 cfs-ft per sq mi measured by L.W. Burt at Daniels Mill on Park River in early seventies
Max. run-off, Farm R. at New Hartford Jan 1936 Flood = 51.9 cfs-ft per sq mi
Run-off in Second Feet Per Square Mile
Time in Hours
100
90
80
70
60
50
40
30
20
0
5
10
15
20
25

FIG. 11. — COMPARISON OF FLOOD RUN-OFF.

The air temperature during this period was quite high, night records being at about freezing and day records showing a maximum between 50 and 60. This period has been plotted on Plate III, and it is interesting to note the variation in discharge following diurnal changes in temperature.

As an indication of what excessive rainfall rates may be expected, the following tabulation is presented for the 10 years of record at the Nepaug dam station:

TABLE 6.

EXCESSIVE RAINFALL RATES.

(From Recording Gage near Nepaug Dam.)

	AMOUNT OF PRECIPITATION IN INCHES.			REMARKS.
	5 min.	10 min.	1 hr.	
June 1, 1914	0.25	0.25	0.40	
Aug. 17, 1914	0.25	0.38	0.51	
Aug. 21, 1914	0.14	0.34	0.59	
Nov. 15, 1914	....	....	0.62	
Feb. 24, 1915	0.06	0.11	0.42	
Apr. 30, 1915	0.17	0.21	....	
May 21, 1915	0.14	0.20	0.75	
July 1, 1915	0.11	0.18	0.50	
July 19, 1915	0.40	0.52	0.58	20 min., whole shower.
Aug. 4, 1915	....	....	1.02	3.5 in. in 6 hrs.
Sept. 21, 1915	0.40	0.61	1.26	2.7 in. in 4 hrs.
Aug. 23, 1916	0.30	0.55	1.42	1 in. in 22 min.
July 17, 1918	0.65	0.95	1.12	1 in. in 11 min., 15 min., whole shower
May 21, 1918	....	....	....	4.5 in. in 4 hrs. at East Branch.
May 21, 1919	0.17	0.25	0.55	
Oct. 16, 1919	0.15	0.22	....	0.65 storm.
Sept. 11, 1919	0.14	0.30	....	0.59 storm.
May 21, 1920	0.09	0.16	....	1.19 storm.
June 17, 1920	0.07	0.10	0.38	2.68 storm.
Sept. 12, 1920	0.20	0.30	....	0.52 storm.
Sept. 28, 1920	0.13	0.22	....	0.62 storm.
Sept. 30, 1920	0.12	0.22	0.52	3.27 storm.
June 29, 1921	0.50	0.90	1.24	3.18 storm.
Sept. 30, 1921	0.38	0.62	....	0.93 storm.
July 3, 1922	0.13	0.34	0.40	1.20 storm.
Aug. 2, 1922	0.12	0.16	0.21	0.90 storm.
Oct. 8, 1922	0.21	0.45	1.46	2.81 storm.
Oct. 23, 1922	0.15	0.20	0.44	0.83 storm.
Jan. 1, 1923	0.08	0.13	0.32	1.31 storm.
April 28, 1923	0.10	0.15	0.50	2.30 storm.
June 4, 1923	0.19	0.27	....	0.44 storm.
April 6–7, 1924	0.07	0.11	0.54	4.24 storm.
April 18, 1924	0.06	0.11	0.29	1.75 storm.
May 11–12, 1924	0.07	0.10	0.40	1.95 storm.

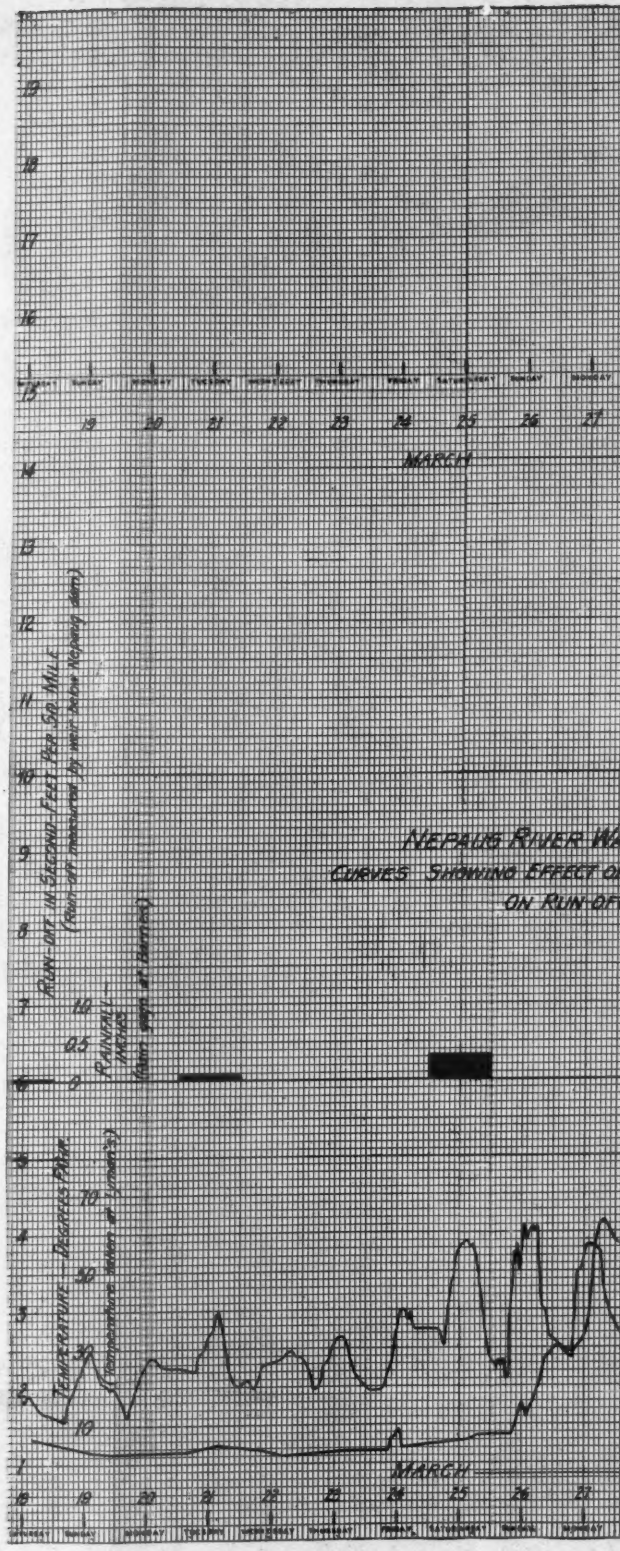
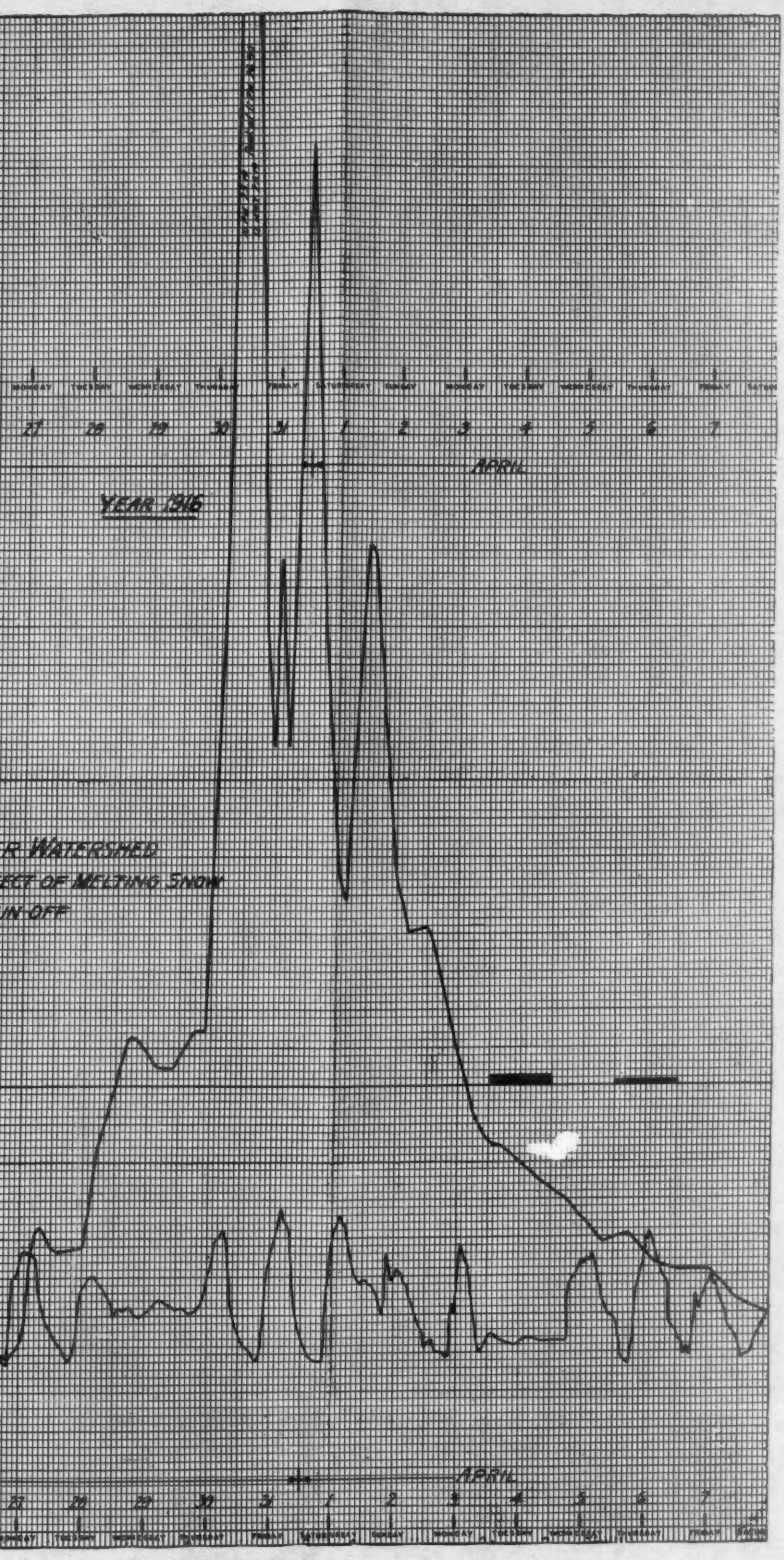

MARCH
MARCH
Run off in Second Feet per Sq. Mile
(from observations by the Boston Water Power Co.)
Rainfall
(from observations by the Boston Water Power Co.)
Temperature, Degrees Fahr.
(from observations by the Boston Water Power Co.)

PLATE III. — CURVES SHOWING



ING EFFECT OF MELTING SNOW ON RUN-OFF.

The following tables also have been abstracted from the report of Messrs. Rudolph Hering, civil engineer, and L. W. Burt, city surveyor, concerning Park River sewage and the general sewerage system of the city of Hartford, July 10, 1893:

Watershed.	Area. Sq. Mi.	Discharge, Cu. Ft. per Sec. per Sq. Mi.	Year.	Authority.
Flathead, R. I.....	61	120.85	1843	J. H. Shedd
Stony Brook, Mass...	12.7	121.03	1886	Com. of Engineers.
Watuppa Lake, Mass.	28.5	72.0	1875	Wm. Rotch
Sudbury River, Mass.	75.2	44.26	1886	D. FitzGerald
Nashua River, Mass.	109.0	104.5	1848	E. Sawyer
Pawtucket, R. I.....	190	58.8	1867	J. H. Shedd
Tohickon Creek, Pa..	102	110.6	1891	J. E. Codman

From the same report there is a note that L. W. Burt, city surveyor of Hartford, in the late seventies (spring) had measured the discharge over Daniels dam in the Park River and found 72.3 cu. ft. per sec. per sq. mi., and at another time 92.1 cu. ft. per sec. per sq. mi. from 76 sq. miles tributary. For improvement of the Park River Mr. Hering did not consider it safe to assume a maximum discharge of less than 150 cu. ft. per sec. per sq. mile.

EXCESSIVE RAINFALLS IN CONNECTICUT.

Place.	Date.	Amount, Inches.	Duration Hr.	Duration Min.	Rate, Inches Per Hours.
Southington.....	June 29, 1879	1.45	0	15	5.80
New Haven.....	July 23, 1889	1.34	0	25	3.22
Canton.....	July 25, 1892	1.33	0	30	2.66
New Haven.....	July 4, 1889	1.07	0	25	2.57
New London.....	July 21, 1885	1.49	0	48	1.86
New Haven.....	Aug. 21, 1888	5.12	3	35	1.43
West Simsbury.....	July 25, 1892	1.07	0	45	1.43
New Haven.....	Sept. 6, 1891	1.35	1	00	1.35
New Haven.....	Oct. 19, 1890	1.18	1	00	1.18
Trumbull.....	Aug. 7–8, 1874	7.30	6	15	1.17
New London.....	June 24, 1882	1.69	1	30	1.13
New London.....	Aug. 21, 1888	0.56	0	30	1.12
Colebrook.....	Aug. 4, 1878	6.67	6	00	1.11
New London.....	Aug. 28, 1891	1.00	1	00	1.00
Voluntown.....	July 8, 1892	1.00	1	00	1.00
Hartford.....	Aug. 18, 1877	3.45	6	20	*

* Much the larger part fell in one hour.

In the report for October, 1913, of the Climatological Service, North Atlantic States, in commenting on heavy rainstorms of 1913 in New York City July 28, September 4–5, October 1–2 and October 24–25, the state-

ment is made that "as nearly as can be determined from the records available, storms of equal intensity have not occurred in the preceding 45 years (a half century at least at the present time, C.M.S.), but since 1869 storms of nearly equal intensity occurred in 1878, 1902, 1905, and 1913. So it is fair to assume that such storms are liable to occur at least once in each ten years. It should be borne in mind that several such storms may occur in one year, and within those of the last twelve years there have been six such storms."

The storm of October 1–2, 1913, was comparatively widespread and recorded in the area to the west as follows:

New York Weather Bureau, Battery Place	5.19 in.
Brooklyn Eagle Office	4.59 in.
Central Park	5.11 in.
Staten Island	8.47 in.
Jersey City, N. J.	6.13 in.
Bayonne, N. J.	7.00 in.
Newark, N. J.	4.40 in.

The rainfall in these storms for various periods was as follows:*

1913.		Maximum Amounts in Short Periods (Inches).						
		Minutes.				Hours.		
		5	10	15	30	1	2	24
July 28	Central Park	0.52	0.99	1.21	1.49	2.74	3.11	3.11
Sept. 4–5	Central Park	0.60	1.45	1.59	2.62	3.31	3.70	4.14
Oct. 1–2	Central Park	0.56	0.78	1.01	1.56	2.03	3.35	5.07
Oct. 24–25	Central Park	0.22	0.37	0.47	0.71	0.83	1.47	3.94

For other storms that have produced floods in New England streams, the following rainfall records are presented, abstracted from "Floods in New England Rivers"†:

July 13, 1897.	Inches.	April 27, 1854	Inches.
Bridgeport, Conn.	8.11	Springfield, Mass.	5.16
Canton, Conn.	5.18	Southwick, Mass.	7.52
Middletown, Conn.	5.09	Saybrook, Conn.	6.00
New Haven, Conn.	5.91	Pomfret, Conn.	3.45
Southington, Conn.	6.60	Concord, N. H.	3.50
Windsor, Conn.	6.90	New York City, N. Y.	4.67
Waterbury, Conn.	6.00	Philadelphia, Pa.	4.87

The storm of October 24–25, 1913, commented on above as giving the greatest precipitation since the establishment of the Nepaug station in 1912 (6.7 in. in 25½ hours), has not been exceeded to date (1924).

George Case, the Weather Bureau observer at Canton, Conn., whose

* Table on p. 4, Report for Oct., 1913, Climatological Service, Dist. No. 1, North Atlantic States, U. S. Department of Agriculture.

† Alfred J. Henry, *Monthly Weather Review*, Dec., 1914.

records go back to 1858, stated that this storm of about fifty hours' duration was the heaviest October storm since 1869.

It appears from the foregoing that there are two types of flood-producing storms to be considered under New England conditions: first, those of rainfall, usually occurring in the fall of the year, and while giving maximum precipitation usually result in less flood flow in the streams, due to the depleted condition of the ground storage after the dry summer period; and second, those occurring in the spring when meteorological conditions bring about a combination of warm rains falling on a snow-and-ice covered country. The floods resulting from these storms are the most severe, although the actual precipitation may be less than that of the fall storms, because of the unlocking of frozen water collected on the ground during the preceding winter months.

During the past century there have been 88 floods of magnitude sufficient to be regarded in the record of the Connecticut River gage at Hartford, Conn. About 40 per cent. of them, as would be expected, have occurred in April and 73 per cent., or about three-quarters of the whole number, in the three spring months, March, April, and May. There have been no river rises of sufficient magnitude to be noted in the month of September, and in spite of the heavy rainstorms which occur in the fall months, October and November, there have been only four times each when excess river height was worthy of notice. (Fig. 12.)

From the diagram itself, if any deduction can be drawn, it would appear that in spite of human developments in cutting down forests and building paved streets and storm sewers, the effect on a large drainage basin is almost negligible.

Since the presentation of this paper additional information has been given in the reports of the U. S. Weather Bureau for 1924 which are abstracted herewith: —

Drought conditions prevailed over all of New England during the entire month of October. The monthly average for the section was 0.44 inches, or 3.06 inches below normal. By States the total precipitation and relation to normal were as follows:

Maine	0.71	–2.75 inches
New Hampshire	0.35	–2.93 inches
Vermont	0.74	–2.45 inches
Massachusetts	0.20	–3.31 inches
Rhode Island	0.34	–3.38 inches
Connecticut	0.30	–3.45 inches

the greatest deficiency being in the southern part of New England.

According to U. S. Weather Bureau records the amount of precipitation during October, 1924, is by far the lowest in the New England section of this same month during the 37 years of record. The next lowest record was that of 1897 with 1.10 inches. For the year 1924 the precipitation and its relation to normal were:

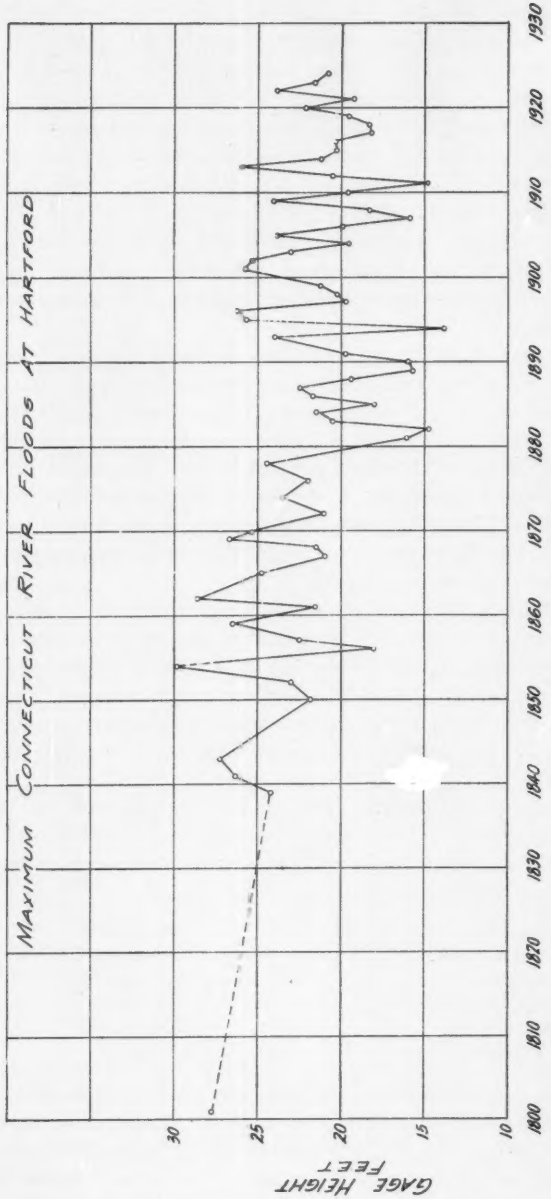


FIG. 12. — MAXIMUM CONNECTICUT RIVER FLOODS.

	New England Section.		Connecticut.	
	Average Precipitation.	Departure.	Average Precipitation.	Departure.
January	3.72	+0.35	4.50	+0.62
February	2.38	−0.83	2.81	−0.92
March	1.47	−2.23	1.60	−2.86
April	4.92	+1.76	6.60	+2.88
May	3.72	+0.47	5.05	+1.63
June	2.02	−1.21	1.98	−1.01
July	2.19	−1.58	1.21	−2.83
August	4.66	+0.89	5.20	+1.07
September	5.45	+1.80	4.64	+0.83
October	0.44	−3.06	0.30	−3.45
November	2.69	−0.66	2.41	−1.20
December	1.96	−1.48	2.27	−1.72
Totals	35.52	−5.27	38.57	−6.96

The Hartford (Conn.) Station of the U. S. Weather Bureau reports in its annual summary a precipitation of 33.54 inches for the year with a deficiency of 11.89 inches. The longest period without measurable precipitation was 33 days, October 9th to November 10th inclusive. The year 1924 also is given as the lowest precipitation of record in the 37 years of record, the highest of record being the year 1920, with 60.96 inches.

As a matter of human health, comfort, and convenience, it is neither desirable nor wise to tamper too much with Nature's provisions for holding the proper balance. Flood discharge, of course, does damage, which must be safeguarded against, as human habitation and industry encroach on the natural storage retardation of the river valley; on the other hand, it is most unsafe and disastrous to carry regulation of the river régime to too great an extent either by storage or abstraction of the flood flows and so bring about uniform conditions of flow. Just as the sediment in a water pipe becomes adjusted to ordinary drafts, and the sudden excess drafts from open hydrants or blow-offs are desirable and necessary to cleanse the pipes, so are the floods of the spring freshets necessary to cleanse the streams of the sewage from cities, wastes from industry, and foulness from natural causes. Too much stream regulation, either by pondage or abstraction of flood flows, is bound to react adversely both on health and industry.

As to whether or not flood conditions are increasing, or otherwise changing, due to natural or human agencies, or both, flood heights in the Connecticut River at Hartford from 1801 to date have been plotted on Fig. 12. for the purpose of putting these data on record. Flood or freshet heights are recorded only when the river gage is at elevation 13 or over.

THE ELECTRIC PUMPING STATION OF THE KENNEBEC WATER DISTRICT AT WATERVILLE, MAINE.

BY ARTHUR L. SHAW.*

[*Read February 10, 1925*]

The Kennebec Water District, in the late summer of 1924, commenced to operate its new electric motor-driven pumping station. In order that the circumstances governing the design of this new project may be fully understood, a brief historical outline of the events leading up to the present situation will perhaps be of interest.

In 1887 the Waterville Water Company, which had been chartered six years previously, constructed a pumping station on the west bank of the Messalonskee Stream in Waterville, and commenced to supply water to Waterville and the Fairfield Village Corporation and to a limited area across the Kennebec River in Winslow, through a distribution system reinforced by a distribution reservoir of about 2.25 million gal. capacity. At this time, the water supply was obtained from the Messalonskee Stream, being pumped into the distribution system by means of water power derived from the same river.

In 1889 the water works were taken over by the Maine Water Company, then owning or controlling numerous water works throughout the State of Maine. This company improved the plant by obtaining additional water rights on the Messalonskee Stream and building a second pumping station across the river from the first.

In 1899, an Act of the Maine Legislature created the Kennebec Water District comprising the territory and people of the city of Waterville and the Fairfield Village Corporation. The District is a quasi-municipal corporation, the affairs of which are administered by a Board of five trustees, two being chosen by the municipal officers of the city of Waterville, two by the municipal officers of the Fairfield Village Corporation, and one, who cannot be a resident of the Water District, by the County Commissioners of Kennebec County. It was not until 1904 that the details and formalities of organization and of acquiring by condemnation the local works of the Maine Water Company were completed, so that the new body could take over the business of supplying water to its constituents.

In 1903, while the water works were still in the hands of the Maine Water Company, a serious epidemic of typhoid was traced to the Messalonskee Stream, so that one of the first acts of the new organization was to seek a new and uncontaminated source of supply. This was found at

*Designing Engineer, Metcalf & Eddy.

China Lake in the towns of China and Vassalboro, where intake works were constructed and from which an aqueduct was built to Waterville. This aqueduct consisted of about 4 miles of 24-in., and about 4.4 miles of 20-in. cast-iron pipe, and was put into service in the year 1905.

At the outset the trustees also made extensive additions to the pumping station on the east bank of the river and a steam pump was installed to supplement those driven by water power.

From these beginnings, down to the present, the water works have been steadily improved, reinforced and extended until a population of about 20 000 is now served in Waterville, Fairfield, Winslow, Benton, and Vassalboro. There are about fifty-five miles of water mains and the average consumption, for 1923, was about 3.1 million gal. per day. Incidentally, the rate has been reduced from 3.75 m.g.d. in 1920, by means of campaigns of inspection to detect waste, by the installation of meters on the services of the largest and most extravagant users, and in spite of the normal extensions of the distribution system and the increase in the number of services since that time.

When the project for a new pumping station was undertaken in 1923, the existing pumping equipment was composed of a 3 m.g.d. Deane compound, condensing, double-acting, duplex pump; a 2.88 m.g.d. Alberger 4-stage centrifugal pump driven by a Hercules hydraulic turbine; and a 2.52 m.g.d. Lea-Courtenay pumping unit composed of two 8-in. single-stage centrifugal pumps connected in series, and driven by 100 h.p. Westinghouse motors. A distribution reservoir, constructed in 1917, has a capacity of 22 m.g., and provides a reserve of about a week's supply.

The need for a radical change in the mode of operation of the pumping plant first became apparent in 1919, when the Central Maine Power Company began to operate a new hydraulic power station at Rice's Rips on the Messalonskee Stream about four miles above the pumping station of the Kennebec Water District. This new power plant was operated only nine or ten hours each day, resulting in the discharge of water past the Kennebec Water District dam during these hours at a much higher rate than could be utilized by the turbine, with a corresponding shortage of water during the remainder of the day. Neither the China Lake aqueduct, the water power pump, nor the distribution system had sufficient capacity to permit the pumping of the entire day's supply of water during the hours of operation of the electric power station, so that it became necessary to do a considerable part of the pumping by the emergency steam plant, and later, by the electrically driven pumping unit which was installed to meet this emergency.

This situation, with attendant loss of water power and expense of pumping by steam or electricity, led the Kennebec Water District into long controversy and litigation with the Central Maine Power Company, which was ultimately settled out of court by a tri-party agreement, which was reached in 1923, between the Power Company, the District and the

Wyandotte Worsted Company, which also owned certain rights in the privileges effected. By the terms of this agreement, the Central Maine Power Company is given a long-term lease of the water rights in question, and is to furnish to the Kennebec Water District electric power of an agreed amount commensurate with the normal water power estimated to have been available on the District's privileges prior to the operations of the Power Company at Rice's Rips. The Power Company agreed to construct a new dam and power plant, combining the privileges of the Kennebec Water District and the Wyandotte Worsted Company; and will benefit by the diversion to its own uses of power in excess of the agreed amount which it may be able to generate by regulation of the stream in conjunction with its other plants on the river. The District agreed to take power during the hours of off-peak load so far as the limitations of its equipment and pipe system permit.

It was apparent at the outset that neither of the existing pumping stations would adequately lend itself to reconstruction for the installation of an all-electric layout, and it was decided therefore to construct a new pumping station on land adjacent to the existing No. 2, or easterly station, and to abandon the station on the westerly side of Messalonskee Stream, reserving the steam pump in the existing station on the easterly side for emergency use in case of failure of electric power.

The first step in the design of the station was the selection of suitable pumping units, upon which the building dimensions would largely depend. Centrifugal pumps were the obvious choice, as best adapted to motor drive.

Nominal pump capacities of 2.0, 2.5 and 3.0 m.g.d. were selected, to give flexibility of operation. The smallest unit was desired for use in case pumping becomes necessary during the hours of peak load on the lines of the Power Company; and the other two units permit pumping at varying rates up to the capacity of the supply aqueduct, when running alone or in combination with another unit, for ordinary use during off-peak hours on the power lines.

The conditions to be met by centrifugal pumps of the sizes chosen were somewhat difficult, owing to the wide range of total head to be pumped against for varying rates of flow in the suction conduit and distribution system. The pressure on the suction side of the pumps varies, owing to the loss of head in the 8.4 miles of suction conduit, from about 93 ft. head at zero discharge to about 12 ft. suction lift at 5.3 m.g.d., the approximate maximum capacity of the conduit.

In the distribution system the variation is not so great, the pressure varying from about 105 lbs., when no pumps are running, to about 126 lbs. at 5.3 m.g.d. These conditions are shown graphically on Fig. 1, plotted from observations made during the acceptance tests, and substantially bearing out the preliminary calculations upon which was based the call for bids. It will be seen that for the minimum pumping rate of

2.0 m.g.d., the total head is 175 ft., while at 5.3 m.g.d. the total head is 307 ft. To be suited for this installation, therefore, pumps must possess characteristics permitting them to operate with reasonable efficiency either alone at the lower heads, or in combination at the higher heads.

The maximum head indicated the practical necessity for pumping in more than one stage. Proposals were therefore considered for both multi-stage and series-connected single-stage pumps, the latter being finally selected, as the proposal which seemed to best fit the conditions

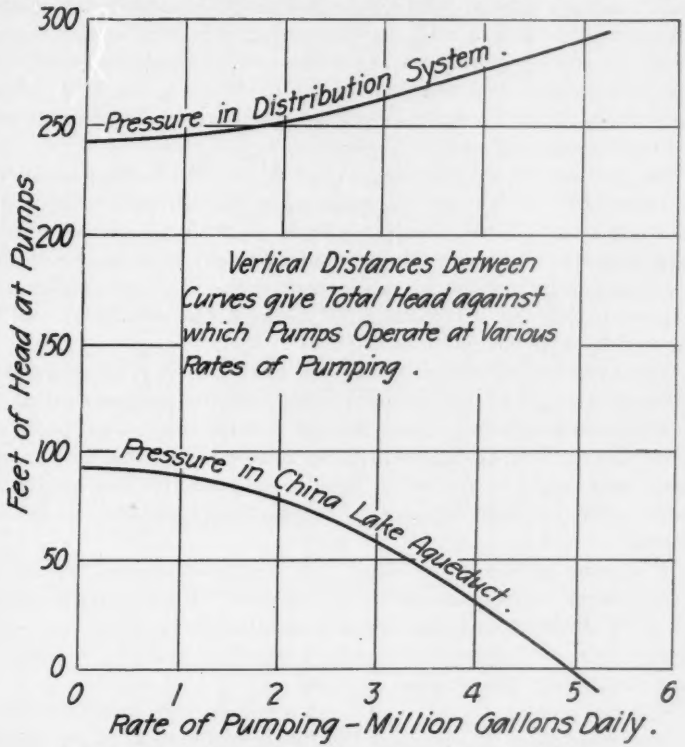


FIG. 1. — RELATION BETWEEN RATE OF PUMPING AND TOTAL PUMPING HEAD.

was based on the use of this type. It was also felt that the use of the relatively simple single-stage pumps, in series, might be of advantage in a somewhat remote location, being free from end thrust and subject to fewer refinements of assembly and adjustment, and therefore more readily maintained in service by the ordinary station operative, without dependence upon the factory expert.

A speed of not exceeding 1 200 r.p.m. was specified, as it was anticipated that gasoline auxiliary prime-movers might be installed on one or

more of the pumps for emergency use in case of failure of electric current. While some engine-builders claim that their gasoline engines can be safely operated at speeds up to 1 750 r.p.m., the more conservative do not recommend in excess of 1 200 r.p.m., for continuous or extended periods. The National Board of Underwriters also recommends the lower figure, although permitting 1 500 r.p.m. as a maximum.

The electric current available is 4 000 volt, 3-phase, 60 cycle, and it was decided to use this current without transforming to a lower, safer, or more common voltage. As will be discussed hereinafter, the station operators will be familiar with the dispatching of current at this voltage through the station switchboard from the new hydro-electric station to the outside lines, so that there is not the objection to the high voltage motors which obtains in some installations where men unfamiliar with high voltages are employed.

The contract for the pumping units was awarded to Starkweather & Broadhurst, Inc., of Boston, who proposed to furnish units composed of two single-stage Morris Machine Works centrifugal double suction pumps connected in series. The 2.0 and 2.5 m.g.d. units have 8-in. suction and discharge, and the 3.0 m.g.d. unit, 10 in. The two smaller units are driven by 200 h.p. squirrel cage 40° General Electric motors, and the larger unit by a similar motor of 250 h.p.

The series piping of each of the two smaller units is so arranged by cross-connection, that the outboard pump may be disconnected at the flexible coupling between pumps, and the inboard single-stage pump discharged directly into the main discharge header. This was done so that a single unit might be run alone, against the relatively low head which obtains under that condition and still maintain approximately its nominal capacity.

The series piping of the larger unit is cross-connected so that the outboard single-stage pump can be driven alone by a future gasoline engine, after disconnecting the motor and inboard pump at the flexible coupling between pumps, thus making available a capacity of about 3.0 m.g.d. should the electric power lines fail.

For the present, the old steam plant will be kept in commission as an emergency stand-by, pending the future installation of the gasoline unit which will be a six-cylinder engine of about 150 h.p., complying with all the Underwriters' requirements for gasoline engines driving fire pumps. Space is provided for a future pumping unit between Unit No. 3 and the end of the building.

The characteristics of the three pumping units as determined during the acceptance tests are shown in Fig. 2. That portion of each curve which is marked "Operating range" includes the rates shown in Table 1 for the corresponding unit, and will be seen to embrace the most desirable section of the efficiency curve.

The lower, dotted curves are the head and h.p. characteristics for the

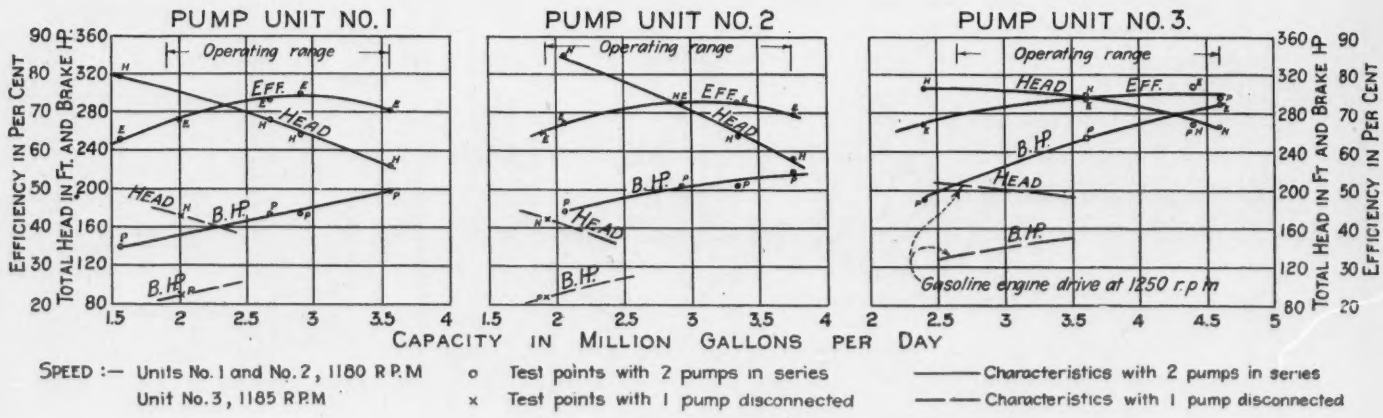
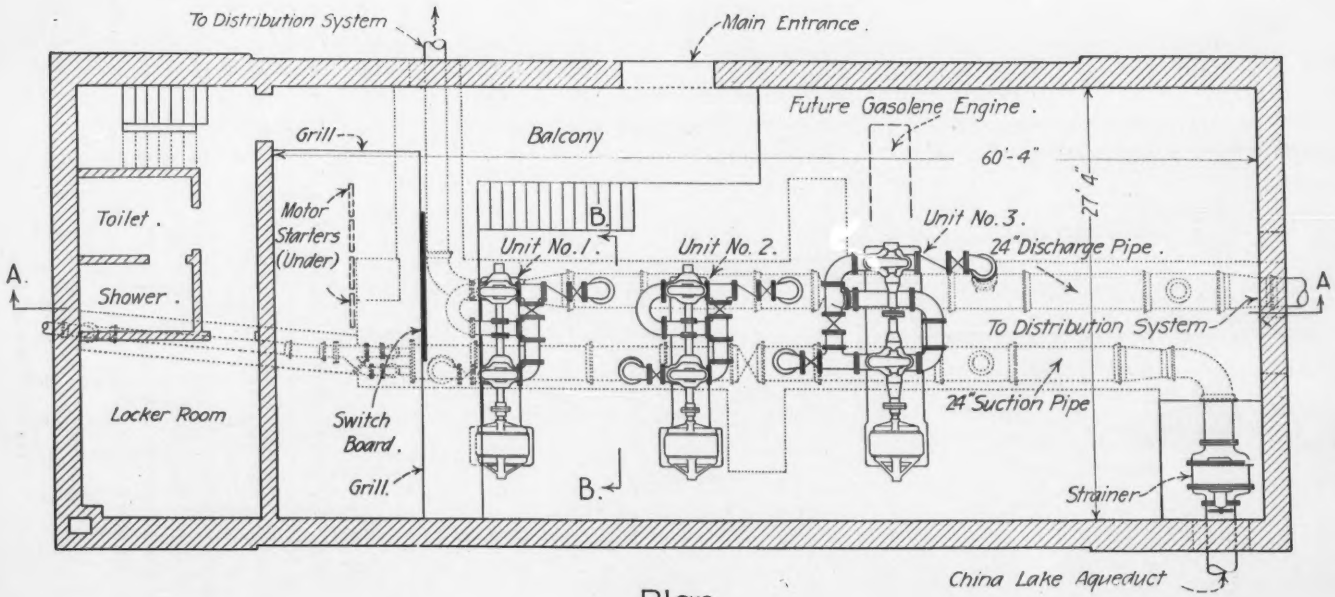
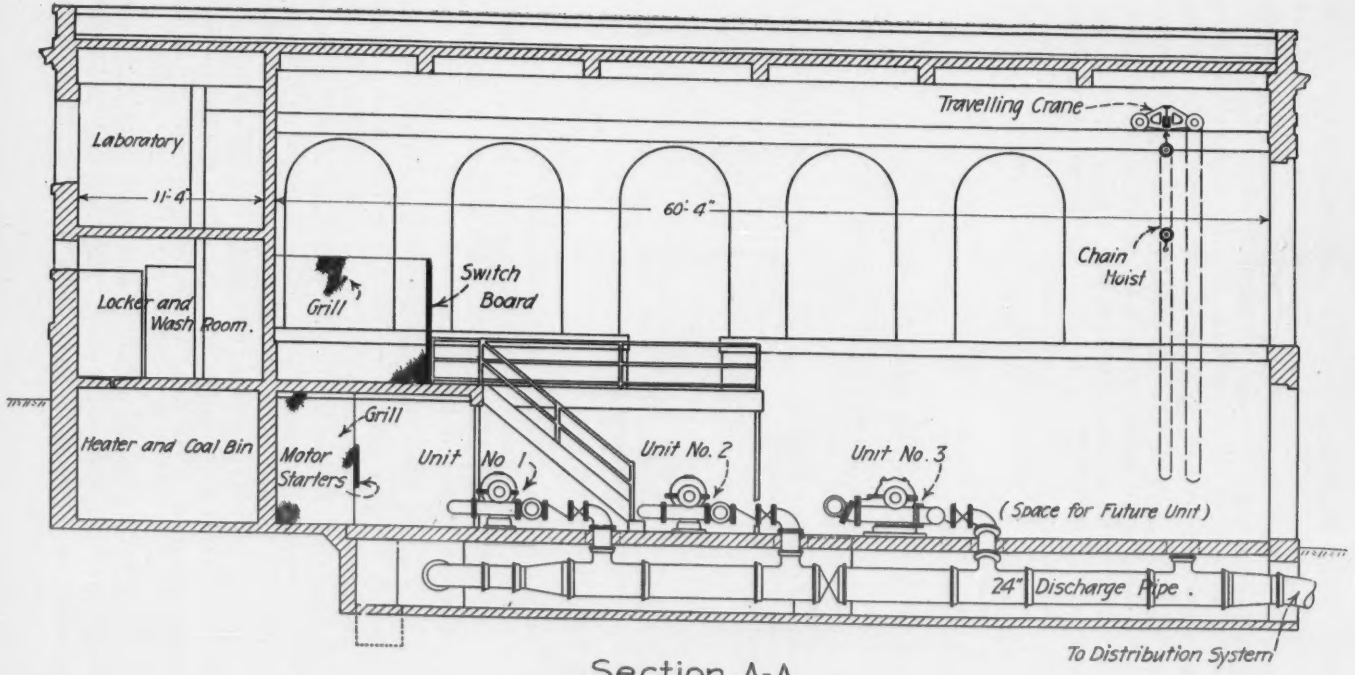

PUMP UNIT NO. 1
PUMP UNIT NO. 2
PUMP UNIT NO. 3.
Operating range
EFF.
HEAD
B.H.P.
Gasoline engine drive at 1250 r.p.m
EFFICIENCY IN PER CENT
TOTAL HEAD IN FT. AND BRAKE H.P.
CAPACITY IN MILLION GALLONS PER DAY
SPEED :— Units No. 1 and No. 2, 1180 R.P.M
Unit No. 3, 1185 R.P.M
o Test points with 2 pumps in series
x Test points with 1 pump disconnected
Characteristics with 2 pumps in series
Characteristics with 1 pump disconnected

FIG. 2. — PUMP CHARACTERISTICS.


To Distribution System
Main Entrance.
Future Gasolene Engine.
Grill
Balcony
60'-4"
27'4"
Toilet.
Motor Starters (Under)
B.
Unit No. 1.
Unit No. 2.
Unit No. 3.
24" Discharge Pipe.
A.
Shower.
A
To Distribution System
Locker Room
Switch Board.
24" Suction Pipe
Grill.
B.
Strainer
China Lake Aqueduct

Plan



Section A-A

FIG. 3. — PLAN OF PUMPING STATION.

units when running single stage with one pump disconnected. In the case of unit No. 3 these lower curves are approximate only, as the gasoline engine has not yet been installed. Motor efficiencies are approximately 94 per cent. at full load.

TABLE 1.

PUMP PERFORMANCE UNDER VARIOUS OPERATING CONDITIONS.

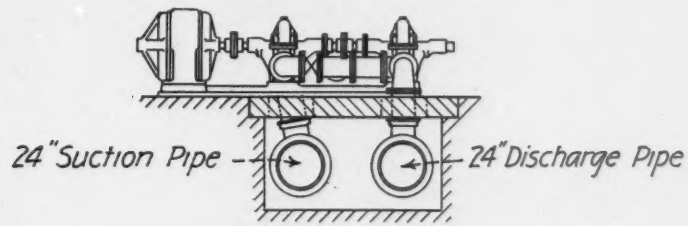
Operating Conditions.	Rate of Discharge M.G.D.	% Pump Efficiency.
PUMPING UNIT NO. 1		
Unit running alone —		
Both stages connected in series	3.56	70.4
Outboard pump disconnected	2.00	68.5
Running with one other unit		
With No. 2	2.1 (4.9*)	70. ±
With No. 3	1.9 (5.1*)	68. ±
PUMPING UNIT NO. 2		
Unit running alone —		
Both stages connected in series	3.75	70.0
Outboard pump disconnected	1.92	65.5
Running with one other unit		
With No. 1	2.8 (4.9*)	73. ±
With No. 3	2.65 (5.3*)	72. ±
PUMPING UNIT NO. 3		
Unit running alone —		
Both stages connected in series	4.59	74.5
Outboard pump, gasoline driven †	3. ±	72. ±
Running with one other unit		
With No. 1	3.2 (5.1*)	72. ±
With No. 2	2.65 (5.3*)	70. ±

*Total from both pumping units when running together.
† No test—engine not installed.

In Table 1 will be found the capacities and pump efficiencies which obtain when the pumping units are run alone or in various combinations against existing head conditions, as fixed by the present suction pipe and distribution system. This table shows the wide range of rates of pumping which is available by operating the units in various combinations without throttling to produce artificial head values. The task of removing or replacing the bolts in the flexible couplings requires but five or ten minutes' time, so that it is a simple matter to shift a unit from series to single-stage operation, or *vice versa*, as the required rate of pumping may dictate.

The pumping units are laid out side by side (Fig. 3), with ample room for the passage of the operator on all sides. The pumps draft from a 24-in. suction header connected with the 20-in. China Lake aqueduct and located

in a pipe tunnel beneath the floor, and discharge into a 24-in. header similarly located and connected with the distribution system at both ends. On the suction conduit just inside the face of the building is located a 20-in. Blackburn-Smith strainer in which are caught the occasional eels, muskrats, or other material which sometimes get into the conduit when the fine screens at the China Lake Intake are removed for cleaning. This strainer has four pail-shaped screening units which can be rotated about a central shaft by means of a lever, permitting any one unit to be brought into a compartment or lock from which it may be removed for cleaning without interrupting the continuity of flow through the other three strainers. This device is located in a conveniently accessible pit and has proven to be useful and practical in operation.



Section · B-B

(see Fig. 3)

At the dead end of the suction header are located two 6-in. Ludlow relief valves, set for about 80 lbs., to relieve the water hammer in the suction conduit when the pumps are shut down. The relief valves discharge into the river. The sudden stopping of the column of moving water in the eight-odd miles of conduit causes a pressure surge which repeatedly travels the length of the conduit. The first, and sometimes the second, oscillation causes the relief valves to open for a few seconds, after which the intensity of the surge quickly drops to about 5 lbs. above and below its normal pressure, and then continues with gradually diminishing intensity until it practically disappears in about 15 minutes. As nearly as can be computed from the data on a recording chart, the velocity of the pressure surge is about 12.5 miles per minute, which happens to be almost exactly the velocity of sound in free air.

A 20-in. by 10-in. Venturi meter is located on the suction conduit just outside the pumping station and the register-indicator-recorder is situated upon the main pump floor. The manually controlled chlorinator is also located on the main floor and feeds into the suction pipe.

A 5-ton hand-operated traveling crane is provided for convenience when repairs or overhauling become necessary. The crane was also used to advantage during erection of the piping and pumping units.

An automatic Schutte-Koerting Universal Water Jet Eductor is installed in a sump in the pipe tunnel to remove accumulations of water from seepage, or other sources, as this tunnel is below the water level in the river.

The architectural details of the pumping station superstructure are by Mr. Horace True Muzzy, Architect, of Waterville. The exterior (Fig. 4)

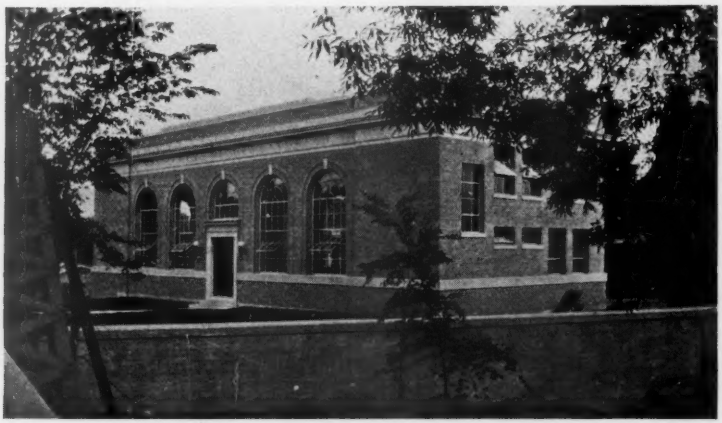


FIG. 4. — EXTERIOR VIEW OF PUMPING STATION.

is of buff brick, trimmed with artificial stone of a light color. In the main pump room (Fig. 5) the floor is of red tile on concrete and the walls are of white enamel tile up to a point about three feet above the gallery level above which buff brick is used. The large windows with steel sash make the pump room interior unusually light. A service entrance upon South Street in the end of the station, at main floor level, facilitates the entry or removal of equipment or large parts.

The pumping station superstructure was erected in the winter of 1923–24 and, to expedite the work, it was completely housed in with rough boarding on the exterior of the staging and heated by steam from the hoisting-engine. It has been stated that the cost of this enclosure was practically offset by the greater efficiency of the masons and other workmen, who worked in comparative comfort; and by the saving in time, no delays being experienced in cold or stormy weather.

The main entrance from Western Avenue, upon which the building fronts, opens upon a gallery above the level of the main pump floor. The main portion of this gallery is devoted to the large main switchboard, the remainder being available as a public space and as means of access to the

laboratory, locker, and wash-room. Beneath the balcony, on the main pump floor, are the switchboards upon which are placed the starters for the motors.

The heating plant and coal bin are located on the same level as the main pump room. Over them, on the gallery level, are situated a locker room, toilet, and shower, and, on the floor above, a laboratory for water analysis. A small area-way, with a hoist of the sidewalk type, permits removal of ashes from the heating boiler at the rear of the building.

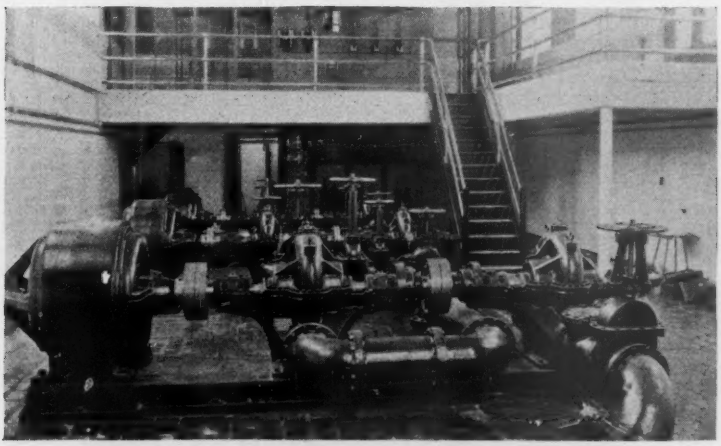


FIG. 5. — INTERIOR VIEW OF PUMPING STATION.

The new hydro-electric plant (Fig. 6), which will be the principal source of power for the new pumping station, is the fourth in a chain of five stations on the Messalonskee Stream, by means of which the Central Maine Power Company is able to utilize to the best advantage the power available on this stream.

It is of interest to note that when the fifth development is completed, with its 1 000 h.p. installation, 175 ft. out of a total of 200 ft. of fall will have been utilized on the Messalonskee Stream between Messalonskee Lake and the Kennebec River, and a total of 10 000 installed h.p. will be available.

Within the dam (Fig. 7), which is of hollow concrete construction on ledge foundation, a pair of S. Morgan Smith Co. horizontal shaft water wheels, attached to a single shaft, drive a General Electric Co. generator, utilizing the combined heads of the two old dams which have been removed. The wheels run at 150 r.p.m., are rated at 1 250 h.p., and have a guaranteed efficiency of 87 per cent. under 23 ft. head at the best gate opening, near three-quarter load. The generator is of 1 000 KVA capacity and produces 60 cycle, 3 phase, 4 wire, 4 100 volt current.

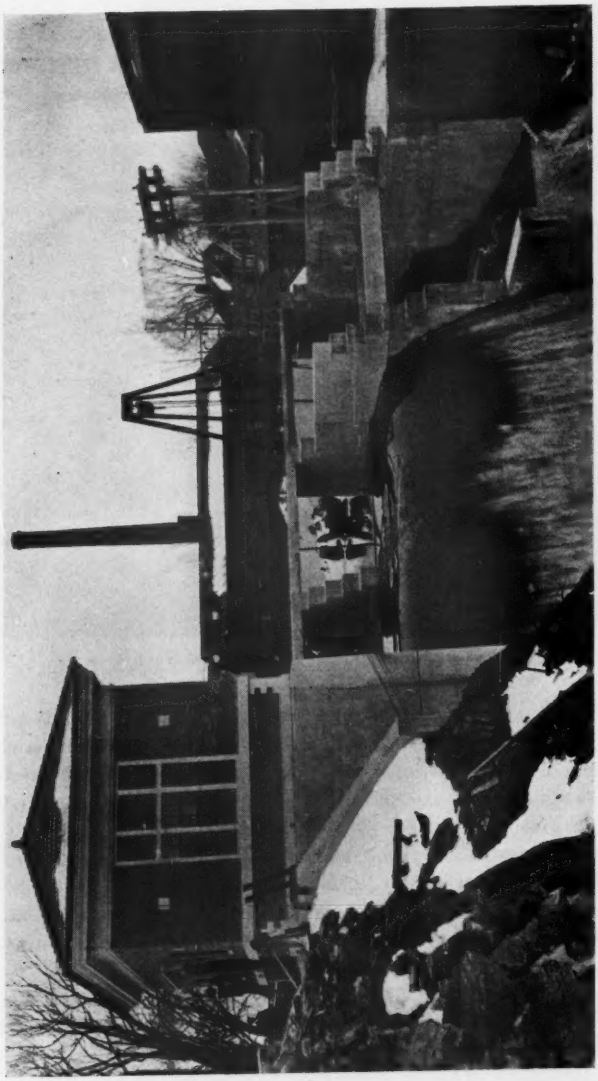


FIG. 6. — CENTRAL MAINE POWER COMPANY'S HYDRO-ELECTRIC PLANT AT WATERVILLE, MAINE.

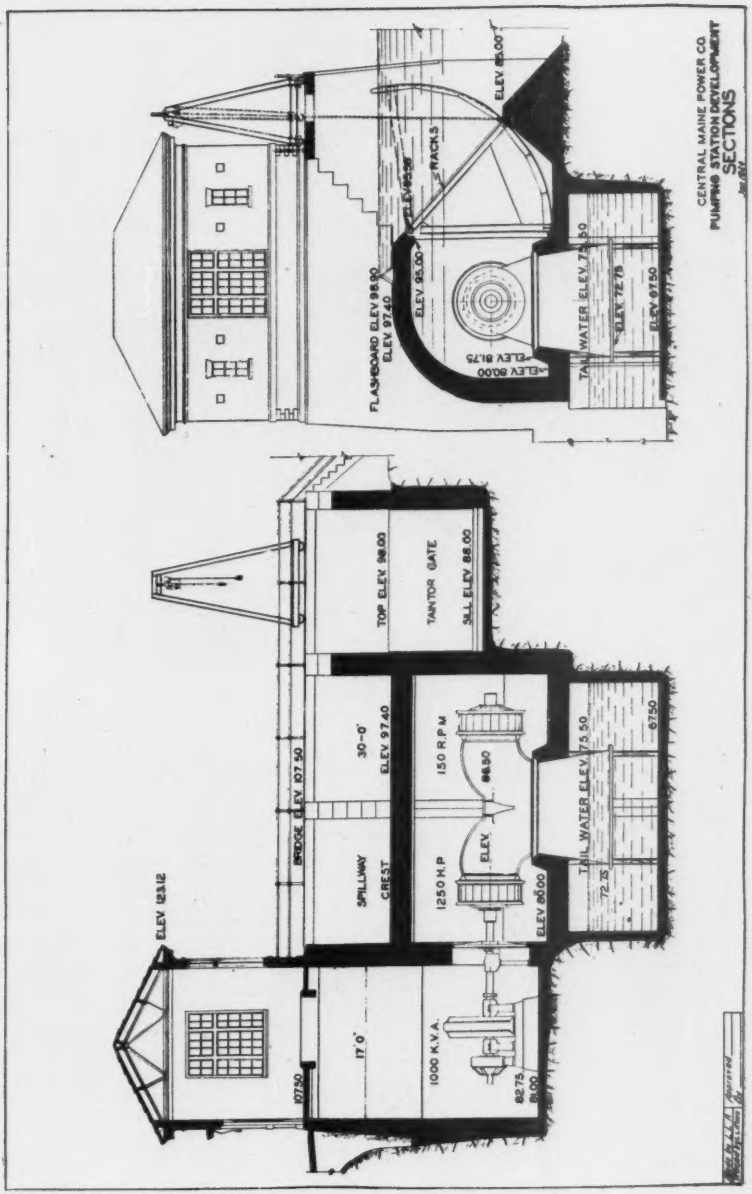


FIG. 7. — DETAILS OF DAM.

This power station is operated automatically by push-button control on the main switchboard in the pumping station, the various steps of opening gates, excitation, and throwing the generator on the line, automatically following one another in proper sequence. Manual operation at the dam is also possible.

Power from the new power station is delivered to the main switchboard (Fig. 8) at the new pumping station through underground conduits, so that the possibility of interruption of power due to ice storms, wind, etc., is reduced to a minimum. Any power not used in the pumping sta-

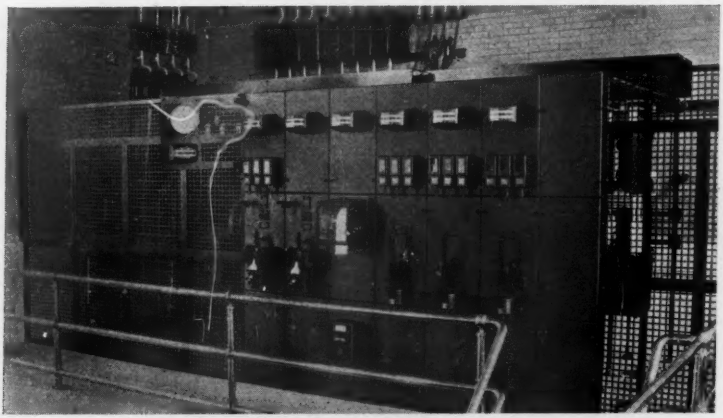


FIG. 8. — MAIN SWITCHBOARD.

tion can be dispatched into the distribution system of the Central Maine Power Company by means of switches and synchronizing devices on the main switchboard at the pumping station. In case power is not available from the new hydro-electric station, the pumping station can be connected with either the Oakland station or the Bangs station of the Central Maine Power Company, so that three sources of electric power are available to meet an emergency.

The new pumping station cost approximately $62 000 for the building, and $20 000 for pumps, switchboard, and other equipment; a total of $82 000, including engineering.

The present Board of Trustees consists of Dr. Frederick C. Thayer, President, Charles E. Warren, L. Eugene Thayer, William W. Nye, and Silas T. Lawry.

Mr. A. B. Thompson is the present superintendent, under whose immediate supervision comes the operation of the new pumping station. Acknowledgment is due Mr. F. H. Mason, Chief Engineer of the Central Maine Power Company, for photographs and data relative to the hydro-electric development.

ADDITIONAL WATER SUPPLY FOR POTTSVILLE WATER COMPANY, POTTSVILLE, PENN.

BY N. J. BEISEL.*

[*Read October 1, 1924.*]

HISTORY.

A brief statement of the physical development of our company and its steady growth is essential information to be considered and given due weight in the determination of extensions to provide for future growth. It is obvious that such information is particular to ourselves and its application pertains to situations similar in principle if not in detail.

It is equally necessary for us, connected with the smaller water companies, to insist that the principles of sound engineering economics be applied in determining our extensions as it is for those controlling the large city supplies to do so.

The Pottsville Water Company was incorporated in 1834 by the Commonwealth of Pennsylvania granting it a perpetual charter, to serve its territory, and conferring upon it the right to exercise the power of "eminent domain" in acquiring lands for water supply purposes.

Figure 1 shows its growth, as defined by the number of services, from the year 1836 when it first commenced business. This growth has been rather uniform. There are but two years in its life, 1873 and 1912, when the number of services decreased from the preceding year. In every other year there was an increase in the number of services.

The first source of supply in 1836 was a small spring, and in 1847 another small spring was tapped, both within the town limits. In 1854 these sources were insufficient for the 600 services, and the company was compelled to go to the mountains for its supply, building a small 5 m.g. storage reservoir on Wolf Creek and a small 5 m.g. distribution reservoir within the town, both of which remain in service. In addition to the small reservoir on Wolf Creek, a small intake was established on Mill Creek which controlled a watershed with an area of 20 sq. miles and possessed exceptionally favorable features for future development. However, the company was forced to abandon Mill Creek in 1860, due to its pollution by anthracite coal mining. The court ruled, when this pollution of a stream being used for a domestic water supply was brought before it by our company, that: "The mining companies possess the right to discharge mine drainage into the natural watercourses."

This fact is of extreme importance in our district, for it necessarily restricts the sources of domestic water supply. More recent decisions on

*General Manager and Chief Engineer.

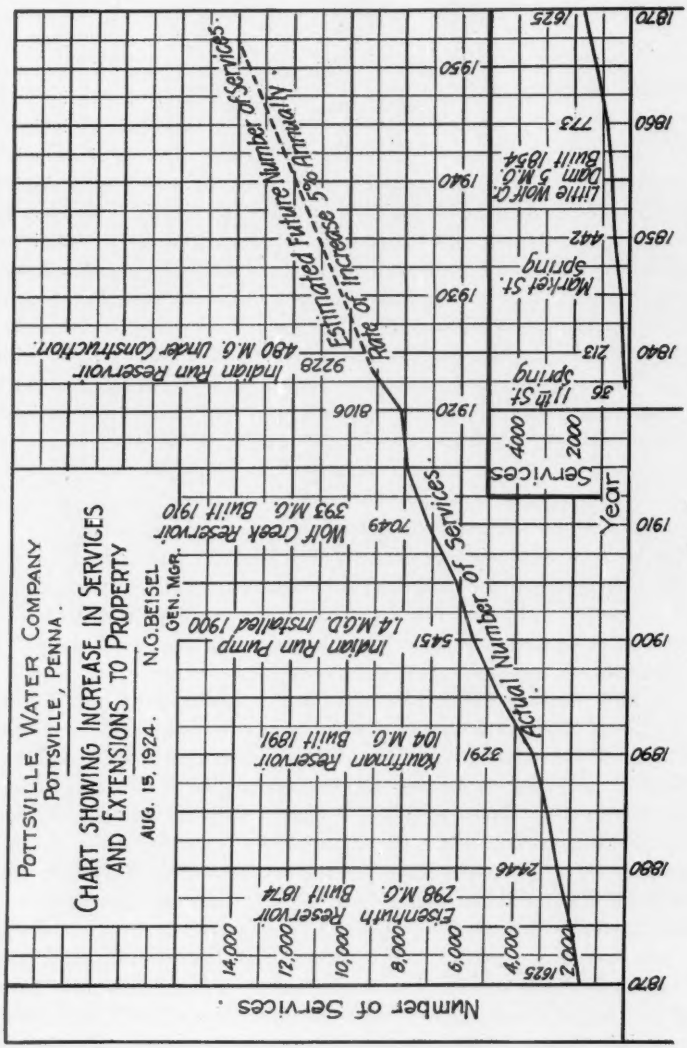

POTTSVILLE WATER COMPANY
POTTSVILLE, PENNA.
CHART SHOWING INCREASE IN SERVICES AND EXTENSIONS TO PROPERTY
AUG. 15, 1924. N.G. BEISEL GEN. MGR.
Number of Services
Year
Services
Actual Number of Services
Estimated Future Number of Services
Rate of Increase 5% Annually
Eisenhuth Reservoir 298 M.G. Built 1874
Kauffman Reservoir 104 M.G. Built 1891
Indian Run Pump 1.4 M.G.D. Installed 1900
Wolf Creek Reservoir 393 M.G. Built 1910
Indian Run Reservoir 480 M.G. Under Construction
11th St Spring
Market St. Spring
Little Wolf Cr. Dam 5 M.G. Built 1854
1625
2446
3291
5451
7049
8106
9228
213
442
773
1870
1880
1890
1900
1910
1920
1930
1940
1950
1860
1840
1850
2,000
4,000
6,000
8,000
10,000
12,000
14,000
2000
4000
36

Fig. 1.

the same question are the "Sanderson Case," Pennsylvania Coal Company *vs.* Sanderson, 113 Pa. 126, in which after much litigation the Pennsylvania Supreme Court held:

"That one operating a coal mine in the ordinary and usual manner, may, upon his own lands, drain or pump the water which percolates into his mine, into a stream which forms the natural drainage of the basin in which the mine is situated, although the quantity of the water may thereby be increased and its quality so affected as to render it totally unfit for domestic purposes by the lower riparian owners, and that damages resulting to another, from the natural and lawful use of his land by the owner thereof in such a way, are, in the absence of malice or negligence, damnum absque injuria."

This same principle is amplified in the so-called "Melcroft Case," Court of Common Pleas, Fayette County, Penn., No. 1023 in Equity, in which, among other rulings, the court held, in paragraph 16 of its conclusions of law:

"The exercise of the power of eminent domain by any of the corporations connected with this litigation having such powers, is not complete and does not constitute the devotion of the waters in Indian Creek to a public use, as against any of the defendants whose property is alleged to have been taken, injured or destroyed, unless and until compensation has been paid or secured to the owner for the injury occasioned by such taking, injury or destruction."

The result of this decision, should it be upheld by the higher courts, is that no water company is justified in developing a watershed within the boundary limits of a coal measure without first acquiring ownership in the coal itself. Otherwise the water company would be in danger at all times of losing its investment through the pollution of the natural drainage by mining operations. It is obvious that the cost of acquiring the coal on a supposedly favorable watershed is generally beyond the means of a water company, and not to be considered.

In directing attention to Fig. 2 and comparing it with the corresponding period, namely 1902 to 1923, of Fig. 1, I wish to emphasize the fact that the annual increase in services is 5 per cent. and the annual increase in metered consumption is 11.1 per cent. This indicates, at least in our case, that consumption does not always vary in direct ratio to the increase in the number of consumers. For the period just mentioned the number of services increased 70 per cent. and metered consumption increased 317 per cent. This is a very important factor and has great bearing in the determination of additional supply. It is especially so in our case, for metered service represents one-third the total consumption. All service in our territory, except domestic, is metered. An interesting feature of Fig. 2, is the line representing "water sold other water companies during drought," and the extreme variation in the quantity. It is apparent this water was needed. Someone had to supply this urgent need and we were

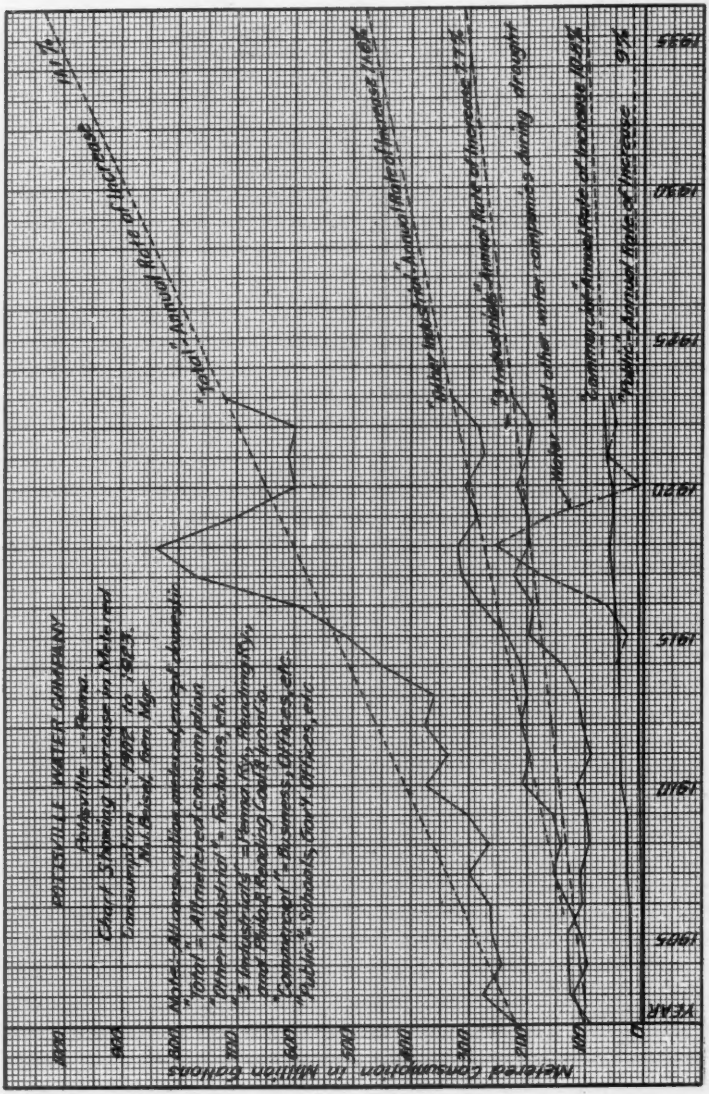


FIG. 2.

the logical parties to do so. It seems reasonable therefore that consideration should be given this feature in attempting to decide upon extensions.

No definite records of total annual consumption for a suitable period of time are available. Venturi tubes were installed on the 16 in. and 20 in. mains in October, 1923, and they indicated an average consumption of 5 m.g.d. Deducting therefrom the metered service, it appears that the domestic consumption was 115 gal. daily per capita.

The population of the communities we serve has increased from 24 475 in 1900 to 35 000 in 1923, and represents an average annual increase of 1.90 per cent. Comparing this with the data on water consumption, we find the following rather interesting facts. For 20-year period 1902–1922:

Average annual increase in population	1.90%
Average annual increase in services	5.00%
Average annual increase in metered consumption	11.10%

It is essential that this variation in consumption increase, compared with service increase, should be noted and careful study given to the facts in the case, to determine its relative weight before deciding upon additional supply to satisfy future demand. An analysis of the metered service chart, Fig. 2, shows an increase in the consumption of water by the individual service itself as well as an increase in the total number of metered services. The three leading industrial services, the Pennsylvania R. R., the Reading Ry., and the Philadelphia & Reading Coal & Iron Company, show, for the past twenty years, a yearly rate of increase in consumption of 7.7 per cent. It seems logical therefore to say it is imperative to recognize these indications in planning additions to storage capacity.

One hundred per cent. metered service is being considered by us as an indirect means of increasing our storage, but it is thought this will not give sufficient relief at this time.

Watersheds and Storage.

Possible sources of supply in our vicinity are limited, due to the pollution of streams by mining companies, as previously mentioned. This compels us to develop the smaller tributaries with drainage areas wholly without the limits of the coal measures, a condition not met in New England, but of great importance in Pennsylvania and other coal or oil mining States. The developed watersheds are:

Name.	Area of Watershed.	Reservoir. Capacity.	Kind.	Year Built.
Eisenhuth	1.64 sq. mi.	298 m.g.	Earth Embkt.	1874
Kauffman	0.86 sq. mi.	104 m.g.	Earth Embkt.	1895
Wolf Creek	1.85 sq. mi.	393 m.g.	Earth Embkt.	1910
Totals	4.35 sq. mi.	795 m.g.		

These watersheds are uncultivated and covered with rather dense brush and second growth timber. Practically all of the land in the various

watersheds is owned by the company. The two undeveloped watersheds under our ownership are Indian Run, with an area of 2.35 sq. miles, and Tar Run with an area of 0.84 sq. mile. No run-off records of these two streams are available prior to April, 1923, when weirs were established thereon to ascertain this information. The statement following gives these data for this limited period, which is too short for definite conclusions.

Month.	Precipitation. Inches.	Run-off, Indian Run. m.g.	Run-off, Tar Run. m.g.
Apr. 1923	1.99	60.9	12.7
May 1923	4.05	100.7	45.6
June 1923	1.93	57.3	7.4
July 1923	3.23	25.8	2.8
Aug. 1923	3.07	16.0	1.8
Sept. 1923	5.74	30.7	7.7
Oct. 1923	3.67	46.0	12.3
Nov. 1923	3.08	73.4	26.5
Dec. 1923	4.85	141.0	87.0
Jan. 1924	5.22	204.8	117.6
Feb. 1924	5.16	69.8	20.9
Mar. 1924	2.01	107.4	52.2
	44.00	933.8	394.5

This record indicates that with an annual precipitation of 44 in. the yearly run-off from Indian Run is 397 m.g. per sq. mile, or 1.09 m.g.d. per sq. mile, which is 52 per cent. of the precipitation, and from Tar Run the yearly run-off is 470 m.g. per sq. mile, or 1.29 m.g.d. per sq. mile, which is 60 per cent. of the precipitation.

Present Storage in Relation to Present Consumption.

Our present storage is 795 m.g. and consumption 5 m.g.d. In the drought of 1922 two of the storage reservoirs were entirely empty and the third contained but 40 m.g., or eight days' supply. There was no other supply available to avoid a water famine. Again, in 1923, in the drought of that year there was in storage 150 m.g., or thirty days' supply. It is obvious our present storage is insufficient for the present consumption.

Fortunately, our consumers, so far, have never been without water or placed upon a daily ration allowance. Sometimes the use of hose for watering and similar uses has been discontinued during droughts. Some of the communities surrounding us have been placed on a daily ration during drought of one to two hours per day. In most every case the plants so rationed were municipally owned.

Lessons to be Drawn from Experience with Present Reservoirs.

Most everyone derives much information and lasting benefit from experiences applicable to the work they are engaged in. It is fitting, I think, to quote extracts from the report Henry P. M. Birkinbine, Civil Engineer of Philadelphia, presented to the Common Councils of St. Clair

and Port Carbon in 1874. It pertains to Eisenhuth Reservoir and was instigated by the citizens being fearful of the breaking of the dam. These towns are part of the territory we serve.

"This reservoir is constructed by throwing a mound of earth across the valley measuring 1 400 ft. on the top. It was intended to be 50 ft. high above the original bed of the creek, with a by-wash 5 ft. below the top of the embankment. From surveys made for the company, at 45 ft. depth, the water would cover 97 acres; at 28-ft. depth, water would cover 55 acres. The storage capacity of the 17-ft. between these depths would be equal to about 400 million gal., and the total storage capacity of the reservoir, when full, probably 700 million gal. The embankment has two 12-in. pipes passing through it for the purpose of drawing off the water, one connecting with the main leading to the distribution reservoir of the works, the other for draining the dam.

"The descent from the dam is very rapid, being over 100 ft. in a mile. The valley is generally very narrow, with steep and rocky sides.

"The construction of the reservoir was commenced in August, 1871. It has cost the company about $75 000. The work was done entirely by men employed directly by the company by the day, and no part of it was built under contract.

"The entire surface covered by the embankment was first cleared of stumps and roots, a puddled ditch was sunk in the center of the embankment 24 ft. wide, diminishing in width as it ascends. This ditch was sunk to a depth of from one to five feet until a satisfactory bottom was reached. The puddle ditch was first filled with selected material. The embankment and puddle ditch were then carried up together in layers of about one foot in thickness. The materials were taken from the sides of the reservoir and are of a satisfactory character. The cross-section of the dam in the bottom of the valley is 168 ft. wide, sloping on the inside one and three-fourths horizontal to one vertical, and on the outside one and one-fourth horizontal to one vertical. The top of the embankment is 39 to 40 ft. wide. The inner slope is protected by a rip-rap of small stones, and the outer slope by a dry wall. The overfall or by-wash is being constructed on the right side of the embankment, and is being excavated out of the hillside. It will be mostly of rock; the side toward the embankment is to be protected by a stone wall laid in cement. The overfall is to be made 30 ft. wide and 5 ft. deep below the top of the embankment.

"There are two 12-in. pipes laid through the embankment; near the bottom they are laid upon and enclosed in masonry. Wing walls are also provided to prevent the water finding its way along the masonry. These pipes are both arranged to be used as drain pipes; one will also be connected with the main carrying the water to the town.

"No engineer has been employed in the construction of the dam, nor had complete drawings or specifications been prepared. The company relied upon the experience which their officers had previously acquired in building dams to guide them in the erection of this one. The stream had not been measured nor had the watershed been surveyed. The above facts will be of importance in this investigation in determining the proper capacity of the drain pipe; and the size of the overflow or by-wash.

"In constructing the embankment I am assured care was taken to make it compact by carting over it, although it would have been better to have built it up in thinner layers, and to have employed rollers as well as carts in making it firm.

"Longer slopes to the inside and outside of the embankment would have been preferable, but if the dam is not carried up higher than it is at present, there is sufficient weight of material to insure its stability. The material is of character to make a good, strong, tight embankment. The rip-rap of small stones on the inside will protect the structure from the action of the waves, if kept in repair. The dry wall on the outside will protect the embankment from the action of the elements, but on account of the acute angle at which it is laid some of it is already pressed out of position.

"The puddle wall is 24 ft. wide at the bottom, tapering to 4 ft. wide at the top, as the reservoir was intended to be finished. The trench to receive it was excavated to a depth of from one to five feet. The officers of the company state that it is entirely placed upon firm foundation impervious to water. The material shown to me as that used in the puddle wall would made a perfectly tight wall.

"The pipes leading through the embankment are entirely too small for emptying the reservoir. This was made evident in several instances. In March, 1873, a large amount of snow had accumulated on the mountain, which was quickly melted by a warm rain, when, notwithstanding both pipes were open, the water rose and flowed over the embankment in considerable volume for some time. The two pipes under this head, if unobstructed, would pass about 1 m.g. per hour, or from 20 to 25 m.g.d.

"If, therefore, the reservoir was full to the present overflow line, it would require about a month to empty it with both pipes open. These pipes should have been very much larger to make it possible to drain the reservoir in time to be of any use in case of emergency. The pipes are properly laid and carefully secured.

"Much has been said, and conflicting statements made of the number and location of springs under the embankment, and the manner in which they were dealt with. The officers of the company assert that there is but one spring under the embankment marked upon the drawing, situated about 300 ft. south of the creek and about 12 ft. from the inside of the embankment. The spring was deep seated, as is evidenced by the temperature of the water. It was followed a short distance into the hill, was covered with stone, and drained into the dam. Springs are always troublesome, and have frequently proved disastrous when under such embankments. Without a personal inspection of this one it is impossible to form an opinion of its character. The officers of the company appear to be satisfied that there is no danger to be apprehended from this spring.

"From all the information collected, the dam appears to be well built, and a reliable structure. But on account of the disastrous results that might follow its bursting, no precaution which will add to its safety or guard against any possible accident should be neglected.

"The breadth of the overfall should be increased to at least 60 ft. to meet the requirements of storms which may fall upon the area drained into the dam, as shown above."

The pertinent points of Mr. Birkinbine's report that should be noted, are: Width of embankment crest 40 ft., which appears excessive for this height (33 ft.), but actually is not, the base being but 168 ft.; the method adopted in disposing of the discharge from the spring under the embankment, namely, draining it into the reservoir; the inadequate drain pipe capacity to empty the reservoir, the embankment being overtopped during the construction period, which must have caused some loss, though none is mentioned; and the fact that the reservoir site was not grubbed, which affected the quality of the water, as noted later.

After this report was made, the company built a second spillway at the opposite end of the dam and also placed a 30-in. cast-iron pipe 12 ft. below the flow line as an emergency outlet. In 1883, the seepage from this dam was measured and found to be 138 000 gal. per day. At present, with reservoir full, it ranges from 130 000 g.p.d. to 217 000 g.p.d., as measured by a V notch weir, the variation depending upon the precipitation on the area between the weir and crest of embankment, which is three acres. No signs of discoloration in the seepage are visible. The embankment is

FIG. 3. — KAUFFMAN DAM.

thoroughly saturated and water can be seen trickling through the downstream tow of the embankment underneath the slope paving.

It is interesting to note that in 1880, six years after this reservoir was first placed in service, the consumers complained of a woody taste and bad odor to the water, and requested it to be remedied. The reservoir site was not grubbed and cleared of vegetation, although the embankment site was. This, no doubt, accounts for the complaint of the consumers.

Another occurrence of value to us, though costly, but unattended by the loss of life and property, was the failure of Kauffman Reservoir. This dam was originally built similar to Eisenhuth and completed in 1886. On June 21, 1892, it alarmed everyone by evidence of failure, and the people below it left their homes and camped on the hillsides. The photograph (Fig. 3) shown was taken on June 24, three days after appearance of the first leak, and the remarkable feature is that, with such exceptional leakage through the embankment, it did not break, but the reservoir of 100 m.g. capacity and 50 ft. head emptied through the crevices in the embankment material and the 2 12-in. outlet pipes without a complete rupture of the embankment.

J. James Croes, of New York, in August, 1892, reports on this failure as follows:

"After examination of the Pottsville Dam at Kauffman's Run, which gave way in the summer of 1892, I am of the opinion that the failure of the embankment was due to the character of the material used in its construction and the manner in which the dam was built.

"On examination of the face of the pits which are now in course of excavation in that portion of the bank which has slid, it is evident that the embankment was made of the earth from the hillsides in the vicinity, in which there was a very large proportion of clay, of partially decomposed red shale, and of muck from pit-holes containing a great deal of fibrous vegetable matter.

"These materials had evidently been carted onto the bank irregularly, dumped and spread, and then consolidated by being carted over, but not by being rammed or rolled.

"The bank appears to be pretty compactly built, but on analysis of the material as it stands now in the bank I find it will absorb 25 per cent. of its bulk of water. A mass of the material put into water falls to pieces, and when this is stirred up and allowed to settle, the heavier particles fall to the bottom and the lighter materials settle on the top, and the whole mass as settled contains just 50 per cent. of its bulk of water. The upper 50 per cent. of the mass thus settled is so fine that it is held in suspension in the water and flows like quicksand.

"As it stands in the bank, this material weighs about 125 lbs. to the cubic foot, or just twice the weight of the same bulk of water.

"I was informed that in the center of the embankment there had been built a puddle wall composed of the same material as the rest of the embankment, but more thoroughly incorporated by being wet and cut during construction.

"The result of such treatment of this material would be to make it more compact and less pervious to water, but I did not see any material on the ground which would make a good puddle wall sufficiently impervious to water to be trusted to make a dam of the size of this one.

"The cause of the failure of the dam is manifest.

"The bank built contained 25 per cent. of void spaces. The water in the reservoir

gradually filled up these spaces and percolated through the bank until it reached the outer face. As soon as this outer face was reached by the percolating water, the very fine material began to wash out. The process was very gradual and for a long time, but it was constant. The washing out of the fine material gradually made waterways, until at last the volume and current of water became so great that it flowed down over the face of the slope wall on the dam below the point of exit. Then the reservoir was drawn down suddenly and the whole inner face of the dam, which was thoroughly saturated with water, slid in as soon as it was relieved of the pressure of the water in the reservoir.

"The complete saturation of the dam is shown by the fact that at the present time, fully a year since the reservoir was entirely emptied, the bank is full of water, which runs out in streams whenever a channel is cut into the body of the bank."

Clemens Hershel of New York in June, 1892, reported on this failure as below:

"I think this is a case of the water following the masonry column which protects the two outlet pipes, and, at the same time, a failure of the central puddle core-wall to stop the water at the point where the masonry column passes through the core-wall. This is always a dangerous point in the construction of such reservoir embankments, and can be guarded against only by ample wing walls projecting from what I have called the masonry column; or, in a better way, by a more efficient kind of core-wall. Whether the earth itself, of the dam, was laid up, spread, watered and rolled, in as efficient a way as should have been done, it will be needless now to inquire. Strictly speaking, the earth itself should form a water-tight and durable dam, and any core structure used should be a force held merely in reserve.

"Be all that as it may have been; in my own practice, I use either a masonry or a concrete core-wall, footed into the ledge or into the strata, such that there is no danger of water passing beneath the core-wall and extending to high-water line, and from side-hill to side-hill. Then lay up the earth either side of the core-wall in a water-tight manner.

"To make your dam water-tight, I recommend that it be reconstructed, so as to be constituted, in its resorted state, according to the general design just stated."

This comprises the general information we had to consider in deciding upon the location and type of project to increase our storage.

Before discussing our new Indian Run development I wish to recite briefly the history of acquiring the lands on this watershed. They were acquired between the years 1891 and 1899.

Eleven hundred and forty-eight acres were purchased for from $20 to $30 per acre. The land is uncultivated and covered with second growth timber and brush. Thirty-four acres at the confluence of Indian Run and the Schuylkill River were condemned by the exercise of the right of eminent domain conferred upon our company by its charter. The jury of view under the County Court awarded the owners $25 000. Both parties appealed from the award. In hearing the appeal, the County Court admitted as evidence, the value the property possessed based on the use of its water for water-power purposes and as a domestic water supply for neighboring communities other than our own. The jury of the County Court hearing the case awarded the owners of the property $45 000 and assessed the costs against the company. The company appealed to the State Supreme Court and the first argument was heard by the seven judges sitting. After a

TABLE I, SAFE YIELD OF INDIAN RUN RESERVOIR

Safe yield per square mile of land surface only, mil. gals. daily	Required Storage in million gallons		Safe yield in million gallons for 2.35 square miles of drainage area with 3% water surface.	
	Per sq. mile drainage area	For 2.35 square miles (entire) drainage area	Daily	Per year
0.5	77	175	1.13	412
0.6	105	237	1.36	496
0.7	153	348	1.59	580
0.8	211	480	1.82	664
0.9	349	792	2.04	745
1.0	517	1172	2.27	829

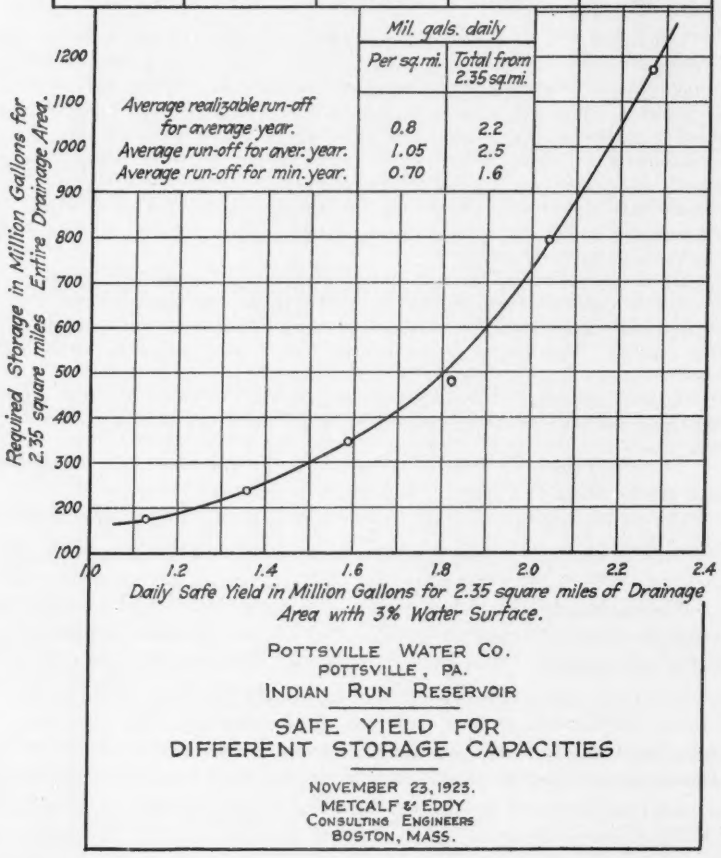


FIG. 4.

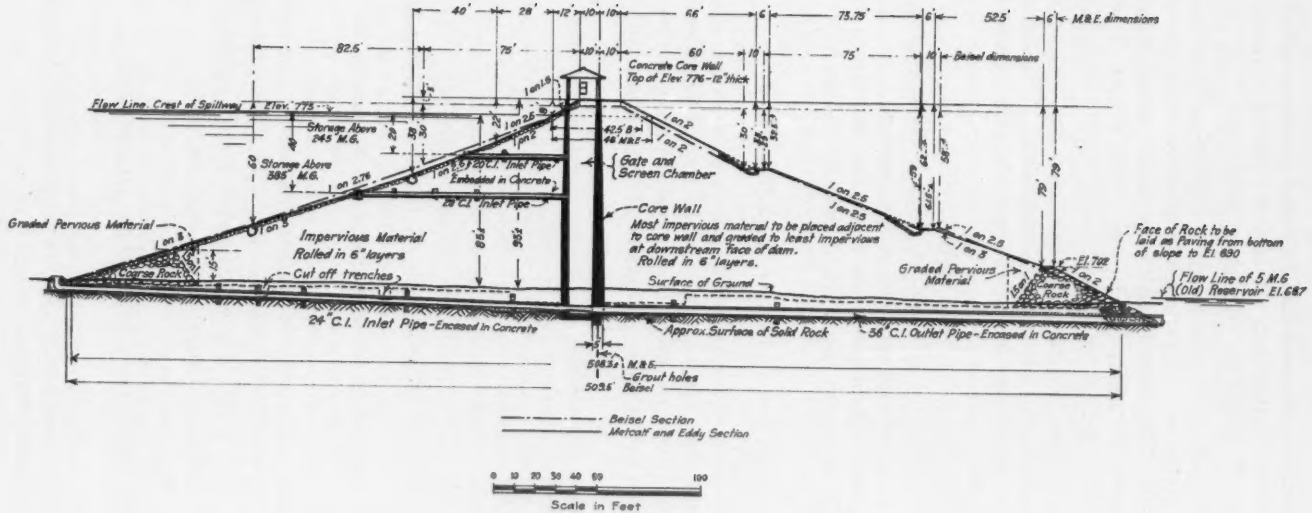


FIG. 5. — PROPOSED CROSS SECTIONS OF INDIAN RUN DAM.

period of one year the Supreme Court ordered a reargument of the case and set a date for such argument. Prior to the date of reargument one of the seven judges who heard the original argument died. The reargument was then heard by six of the original seven justices. This court, after reviewing certain testimony, and certain legal questions affecting our particular problem, held: "This court being *evenly* divided on the argument and evidence presented, we affirm the decision of the learned court below." This placed a value on land for domestic water supply determined by its worth for that particular purpose, although it was not being used for that purpose at the time of condemnation proceedings. The judge of the County Court in his remarks to the trial jury upon the admission of such evidence said, "The jury must put no weight on such testimony or evidence, but they have the right to decide in their minds and to give and award to the owner of the property any sum which they consider proper for the particular use the purchaser wishes to put the property."

The net result of this litigation was that it cost $62 570.32 for 34 acres of this watershed to protect our title and make it unassailable.

New Indian Run Storage Reservoir.

It was determined to develop Indian Run watershed and, after compiling the necessary preliminary information, Metcalf & Eddy were retained as Consulting Engineers to review the general situation, and report on its possibilities, examine our plans, and make the suggestions and corrections necessary.

The embankment was located after investigating several sites by test pits, churn drill and diamond drill borings.

Safe Yield of Drainage Area.

Lacking local records, the estimate of probable safe yield of the drainage area tributary to the company's reservoirs, was based upon existing records of the yields of other streams in Pennsylvania and neighboring States.

The yields assumed for the Indian Run project are shown in Fig. 4.

Height of Dam.

Careful study was given to the height to which the dam could be built economically. It was decided that the crest of the spillway should be fixed at Elev. 773. This would give an estimated storage of 482 m.g. and a minimum safe yield of 1.79 m.g.d. or 655 m.g. per year.

Cross Section of Dam.

In Fig. 5 is shown a cross section of the proposed dam.

Spillway.

The method of designing the spillway is given in the following extract from Metcalf & Eddy's report.

"The spillway must have a capacity corresponding to the discharge from the reservoir under maximum flood conditions. Prudence dictates that the spillway should be given this capacity without allowance for the water which may be discharged through the outlet pipe from the dam, because of the possible danger of the clogging of the latter under extreme flood conditions and the necessity of preventing the overtopping of the dam.

"The experience with the spillway of the old dam upon Indian Run, which is about 30 ft. long by 3 ft. deep and which has already been overtopped, has been too short (about 15 years) to be of significance. It is certain that in a period of 100 years or more, much larger quantities of water will have to be handled. This spillway should be enlarged.

"In our analyses of the problem we have given consideration to the magnitude and character of storms likely to produce the largest run-off and to the record of maximum run-off or discharge realized from a large number of streams in Pennsylvania and other states.

"It is clear that with so small a drainage area as this, 2.35 sq. miles—approximately rectangular in shape, with side slopes of 400 ft. per thousand at the upper end, and 140 ft. per thousand adjacent to the valley; with a slope of the main valley of about 19 ft. per thousand, though somewhat steeper at the lower end; and with no well-defined drainage lines on the side slopes — a short, intense storm will produce a larger flood wave than a heavier rainfall of longer duration but less intensity.

"As an aid to judgment, we have assumed a rainfall of 6 in. in 3½ hours, varying in intensity as follows:

Hours after beginning of rain.	Rain Rate per hour for a 30-min. period.
0:30	2 inches
1:00	3 inches
1:30	3 inches
2:00	2 inches
2:30	1 inches
3:00	0.5 inch
3:30	0.5 inch

"Assuming a velocity of flow of water over the surface of the ground, under storm conditions, of 10 ft. per sec. for the steep slopes, and 5 ft. per sec. for the flatter slopes, the running time from the ridge to the valley would be 6 minutes. To allow for concentration of water and other factors such as absorption, etc., the time for concentration or flow from the watershed, or divide, to the stream may fairly be fixed at 10 minutes. The slope of the channel is about 2 per cent. at a point 1 000 ft. from the divide, and with a run-off of 1 in. per hour, the quantity flowing will be approximately 115 cu. ft. per sec. The velocity of flow in the channel is likely to be about 8 ft. per sec., and the time of flow from the upper end of the drainage area to the reservoir about 20 minutes. The total time for concentration, including flow over the surface of the ground and in the channel of the stream, would then be about 30 minutes.

"Assuming that the total amount of retention will equal 1 in. of rainfall over the entire area in the extreme storm cited; that the greater portion of this retention will occur during the first hour, and that the inflow into the reservoir will be delayed for a period of about 10 minutes after the rainfall causing the inflow; the maximum rate of inflow to the reservoir will be about 3 915 cu. ft. per sec. and occur during the fourth half-hour period. This rate is equivalent to a run-off of approximately 1 700 cu. ft. per sec. per square mile. This amount corresponds fairly with actual run-offs which have been realized in different streams.

"With the foregoing assumptions and the spillway crest assumed 40 ft. long at Elev. 773, it is probable that the water surface will rise 6½ ft. above the crest of the spillway (or to Elev. 779.5); and that the maximum discharge from the spillway will be at the

rate of about 2 500 cu. ft. per sec., equivalent to 1 065 cu. ft. per sec. per sq. mile of drainage area. It will be observed that the storage capacity (113 m.g.) between the crest of the spillway (Elev. 773) and the flood level (Elev. 779.5) has the very important effect of decreasing the outflow from the basin to 2 500 cu. ft. per sec., though the corresponding rate of inflow is about 3 915 cu. ft. per sec. The height of the dam above the flood level in the reservoir (which has been fixed by us at 18 in. and which gives an additional storage of 30 million gal. and makes the entire freeboard or height between crest of spillway and crest of dam 8 ft.) is an additional factor of safety, as is also the capacity of the blow-off pipe through the dam."

Construction of Dam.

Grouting.

The core-wall trench was excavated to solid ledge rock, the bottom width of the trench being 25 ft., side slopes 1:1 and the average depth 14 ft. The stratified rock was then removed by bars and wedges for a minimum width of 5 ft. and a depth of 3 to 5 ft. $2\frac{1}{2}$-in. grout holes, 18 to 24 ft. deep, were then drilled by the ordinary tripod drill into the bottom of this trench and 2-in. grout pipes, 4 ft. long, cemented in place for a grout pipe connection. A small portable 280 cu. ft. compressor supplied the air for the ordinary grout mixer of 3.5 cu. ft. capacity. The pressure applied ranged from 60 lb. to 80 lb., but most of the grout was injected under the higher pressure.

The grout was mixed in the machine in the proportions of 1 part cement to 10 parts of water. No sand was used at any time. Occasionally the mixture was thickened by using 2 to 3 parts cement to 10 parts of water.

Some of the holes discharged water under a slight head before being grouted. The elevation of this water (after coming to rest within the grout pipe when the grout pipe was raised) was 687 and the elevation of the water surface behind the temporary diversion dam was 695, while that in the old pool below the dam was 680. It seems reasonable, therefore, to suppose that the grout holes tapped underground water seams.

It is supposed some dams in our neighborhood leak through the fissured red shale underneath the core-wall on which they are founded, and it is considered by some to be almost impossible to make these fissured strata water-tight. It is impossible to remove it entirely for it lies almost vertical, is between layers of hard sandstone, and extends for approximately 2 000 ft. below sea level at the same inclination of 75° from horizontal.

When making the initial grouting in a hole and the grout appeared upon the surface of the rock through the fissures and seams within a radius usually of 12 ft. from the hole, grouting was stopped and the grout pipe extended up through the concrete core-wall. The core-wall was then built up to the original surface of the ground, about 14 ft. in height, and the open trench backfilled on both sides of the core-wall with material puddled in water from 8 to 10 ft. in height and then with rolled earth embankment on top of this material up to the original surface of the ground and higher. Grouting operations were then resumed in these holes and every one so far tried was grouted to refusal at 80 lb. pressure applied for ten minutes. In

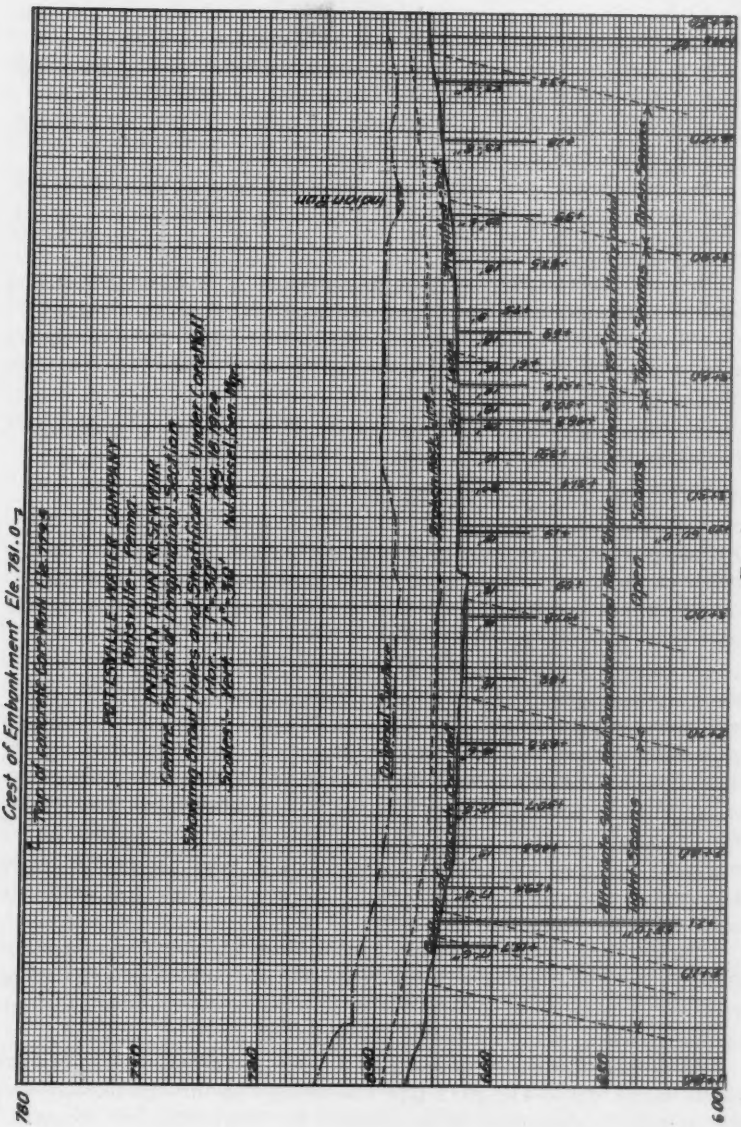

Crest of Embankment Ele. 781.0
Top of concrete Core Wall Ele. 776.5
PETERSVILLE WATER COMPANY
Petersville, Penna.
INDIAN RUN RESERVOIR
Earth Portion of Embankment Section
Showing Bored Holes and Stratification under Lower Toe
Scale - 1" = 20'
780
760
740
720
600

Fig. 6.

some cases it was necessary to make three attempts to close the holes, after each successive lift in the core-wall, for the grout sometimes appeared on the surface of the rolled earth embankment next to the concrete core-wall with 12 ft. of embankment material over the surface of the rock.

A description of the grouting operations of three typical holes will suffice to disclose the method.

Nothing unusual was encountered in this work and the table following is a condensed statement of the holes grouted to date.

The elevation of the central portion of the embankment shows the location of the various grout holes.

Grout Hole Sta. 2–15.7.

Hole is 17 ft. 6 in. deep in hard red sandstone. No water discharging from hole. First grouting, June 28, six batches 1:10, 0.8 cu. yd., was forced in at 70 lb. pressure. Hole refused next batch at 70 lb. pressure applied for seven minutes. No grout appeared on surface. Grout pipe was then extended through core-wall.

On July 3 grouting resumed but hole refused to take any grout at 80 lb. pressure applied for 12 minutes. It was then capped.

Grout Hole Sta. 2–21 (Diamond Drill Hole No. 1).

This hole is 65 ft. deep and is one of the diamond drill prospect holes, 80 ft. deep from the surface. No water discharged from this hole at the surface of the ground, Elev. 692, at any time, but after excavating the core-wall trench to solid rock, Elev. 677, it discharged clear and very cold water at the rate of 2 gal. per minute.

On June 28, 30.1 cu. yd. of 1:10 grout and 1.1 cu. yd. of 2:10 grout were pumped into it under 70 lb. pressure. When pressure was applied muddy water discharged from the holes at sta. 2–29.4 and 2–81.4. There always was a slight flow of water from hole at sta. 2–81.4, but none from hole at sta. 2–29.4. The hole at sta. 2–29.4 was capped for five minutes and then the cap removed and no discharge from this hole occurred thereafter. Water also appeared in hole at sta. 2–15.7 but ceased flowing after one hour's grouting. For two hours during the grouting of this hole, muddy water discharged from hole, sta. 2-81.4, 60 ft. distant, after which it gradually lessened and finally ceased completely.

The grouting of this hole commenced at 8 A.M. and at 2:30 P.M. the time necessary to discharge each batch from the machine lengthened. The grout was then thickened, 2 parts cement to 10 parts water, and after pumping nine batches, no more could be discharged under 70 lb. pressure.

At no time during the grouting of this hole did grout appear upon the surface of the rock. The grout pipe was brought up through the core-wall and on July 3, tried under 80 lb. pressure for 15 minutes, but refused to take any more. Hole capped.

Grout Hole Sta. 2–97.8.

Hole is 18 ft. 0 in. deep. June 26, discharged 3.9 cu. yd. 1:10 grout,

1.6 cu. yd. 2:10 grout, and 0.8 cu. yd. 3:10 grout under 70 lb. pressure. Grout appeared on surface of rock and work was suspended.

Resumed, June 29, discharging 3.1 cu. yd. 1:10 grout, 1.2 cu. yd. 2:10 grout, and 0.6 cu. yd. 3:10 grout, under 60 lb. pressure. Grout appeared on surface and work discontinued.

Again resumed on July 16, discharging 3.5 cu. yd. 1:10 grout and 1.5 cu. yd. 2:10 grout, under 70 lb. pressure. At this operation core-wall trench was backfilled for 12 ft. in height, and grout appeared on surface of embankment adjacent to core-wall, whereupon work was suspended.

Resumed August 15, after extending grout pipe 5 ft. through core-wall, and adding 5 ft. in embankment height both sides of core-wall. Discharged 0.12 cu. yd. 1:10 grout and 0.12 cu. yd. 2:10 grout under 80 lb. pressure. Hole refused more under 80 lb. applied for ten minutes. Hole capped.

SUMMARY OF GROUTING OPERATIONS TO AUGUST 15, 1924.

Location of hole — station.	Depth of hole.	Pressure applied.	Number sacks cement used.	Number batches placed.	Cu. yd. grout placed.	Average number sacks cement per cu. yd. grout.
2–15.7.....	17 ft. 6 in.	70 lb.	2	6	.80	2.50
2–21	65 ft. 0 in.	70–80 lb.	84	243	31.20	2.69
2–29.4.....	17 ft. 8 in.	65–70 lb.	16	40	5.10	3.14
2–40.5.....	18 ft. 0 in.	70–80 lb.	2	4	.50	4.00
2–50.7.....	17 ft. 6 in.	70–80 lb.	8	16	2.10	3.81
2–65.8.....	18 ft. 6 in.	70–80 lb.	3	7	.90	3.33
2–81.4.....	18 ft. 0 in.	70–80 lb.	9	25	3.30	2.73
2–97.8.....	18 ft. 0 in.	70–80 lb.	59	124	16.50	3.58
3–09	18 ft. 0 in.	70–80 lb.	22	46	6.00	3.67
3–20	60 ft. 0 in.	70–80 lb.	193	467	60.80	3.17
3–31.4.....	24 ft. 0 in.	70 lb.	56	140	21.80	2.57
3–39.1.....	18 ft. 0 in.	60–75 lb.	75	143	18.40	4.08
3–50.8.....	18 ft. 0 in.	70 lb.	145	335	39.70	3.65
3–61	12 ft. 0 in.	70 lb.	25	49	6.40	3.91
3–69	18 ft. 0 in.	70 lb.	74	173	22.50	3.29
3–75	9 ft. 0 in.	70 lb.	32	60	7.70	4.16
3–87.5.....	18 ft. 0 in.	70 lb.	45	85	10.90	4.13
3–99	23 ft. 6 in.	70 lb.	49	115	14.80	3.31
4–18	23 ft. 8 in.	70 lb.	37	73	9.40	3.94
			936	2 151	278.80	3.36

NOTE: Proportions used in grout varied from one, two, and three parts cement to ten parts water. No sand used in grout.

Embankment Material.

A mechanical analysis was made by W. S. Pardoe and E. F. Stower, University of Pennsylvania, of material available for rolled earth embankment, and a copy of their analysis follows:

Sample A: From side of test pit 2 ft. below ground surface.

Sample B: From side of test pit 7 ft. below ground surface.

DATA ON TESTS.

	SAMPLE A.		SAMPLE B.	
	Specific Gravity, Le Chatlier.	Weight, lb. per cu. ft.	Specific Gravity, Le Chatlier.	Weight, lb. per cu. ft.
Grain	2.603	162.5	2.603	162.5
Dry sample	1.65	103.2	1.61	100.5
Tamped as received	2.07	129.2	2.075	129.6
Maximum density, tamped	2.14	133.6	2.14	133.6
Untamped	2.015	125.8	1.991	124.3
Percentage of voids in dry sample	36.5%		38.1%	
Percentage of moisture in natural state	8.48%		9.72%	
Silt Test				
Stone and large particles	40.0%		36.9%	
Granular — quick settling	21.5%		28.7%	
Fine settling, 24 hr.	38.5%		34.4%	
Loss by ignition (organic matter)	2.93%		3.04%	
Solubility (boiling 1 hr.)	0.13%		0.06%	
Passing $\frac{1}{4}$ in. mesh	66.0%		46.5%	
Effective size	.0225 mm.		.041 mm.	
Uniformity coefficient	22.9		22.4	
Porosity	36.5%		38.1%	
Transmission coefficient	.000322		.001096	

This material packs very thoroughly under the roller and requires no sprinkling except on very hot days. It is observed that after a hard rain the surface of the rolled embankment absorbs water for a depth of 1.5 in. and no absorption was noted below that depth.

Underdrains.

The overburden overlying the rock underneath the embankment is impervious to water except at the junction of this overburden with the rock line where more or less seepage was noticed in the core-wall trench at this point. A flowing spring was uncovered in the core-wall trench and another in excavating to rock for the 36-in. outlet pipe in the upstream half of the embankment. The spring in the core-wall trench was conducted to a 6-in. cast-iron pipe placed in the center of the core-wall at Elev. 681 and from there conducted underneath the downstream portion of the embankment through an 8-in. pipe.

The spring at the 36-in. outlet pipe in the upstream portion of the embankment was conducted through an 8-in. terra cotta pipe 6 ft. below the surface of the ground to the core-wall, and at that point connected to the 8-in. drain on the downstream side of the core-wall. An 8-in. gate valve with riser pipe is embedded in the concrete wall for the purpose of closing this drain on the upstream side of the core-wall with concrete at the proper time.

The concrete outlet tower is wholly within the embankment and incorporated with the concrete core-wall. The type of connection where the 20-in. inlet pipes enter the tower through the rolled earth embankment at

different elevations is patterned after the one in use at the Phelps Brook Dam, Hartford, Conn., through the courtesy of Caleb M. Saville.

Cost.

The Indian Run improvements will cost about $650 000, or $1 350 per million gal. of storage.

It is instructive to compare this cost with that of the reservoirs now in service, as shown in the following table. The cost of the old reservoirs was obtained from the records.

TABLE OF COMPARATIVE COSTS OF STORAGE.

(Exclusive of Land.)

Name of Reservoir.	Storage Capacity, m.g.	Year built.	Total cost, exclusive of lands.	Cost per million gal. of storage.
Eisenhuth	298	1871–1874	$100 250	$337
Kauffman	104	1893–1895	90 000	865
Wolf Creek	393	1906–1910	241 000	613
Indian Run	480	1924–1925	650 000	1 350

The unit prices being paid the contractor at Indian Run are about three times those paid at Kauffman and Wolf Creek. Cast-iron pipe is about 280 per cent. higher now than the price paid at the time those reservoirs were built.

IMPROVEMENTS IN FILTRATION PROCESS AT WATERTOWN, NEW YORK.

BY F. H. JENNINGS.*

[*Read November 11, 1924.*]

On September 12, 1904, the city of Watertown, N. Y., put into operation a rapid sand filtration plant, whose early operation has been described before this society.† Since that time changes in conditions and demands upon the plant have called for material changes in the plant and equipment, which may be of interest to the members of this Association.

The first decided change was in the underdrainage system. We had considerable trouble from the mixing of the sand and gravel and loosening of the brass strainers. This permitted the sand to run out through the hole where the strainer came out and, in some cases, without any preliminary indication of trouble, until the strainer came completely out and the sand ran through in quantity. In other cases the sand, which had mixed with the gravel, worked through the strainers and plugged up the underdrains to such an extent as to render the washing of the filter ineffectual over a considerable portion of the filter. Similarly, the small air-distributing pipes became plugged with sand so that there was very poor aeration of the filters when washing.

To remedy these conditions the old underdrainage system was torn out and the brass strainers were replaced by simple wrought-iron pipes with small holes pointing downward. We have had no trouble with loosening of these pipes, and the pipes themselves are in good condition after more than 12 years of service. None of them have had to be replaced.

We still had mixing of the sand and gravel, which seemed to be due to the use of air in the washing. We soon abandoned the use of air, increasing the wash water rate from 12 to 18 vertical in. per minute. This has eliminated trouble from mixing of the sand and gravel and gives us a better and cleaner wash than we had before. We were also able to decrease the time of washing, so that the increased rate used only a little more water than the lower rate.

Anyone who has used an alum solution feed, especially when it is necessary to pump the solution, knows the troubles inherent therein, due to corrosion. We had our share of these troubles until we abandoned the solution feed and replaced it by a dry alum feed. This has been very satisfactory and gives us very little trouble. The original dry feed machines

*Superintendent Filtration Plant, Watertown, N. Y.

† Jour. N. E. W. W. Assoc., vol. 22, p. 136.

were water motor driven, while the machine installed in our new chemical dosing station — to be described later — is driven by a variable speed electric motor, which we find preferable.

The consumption of water in the early days of this plant was about 4 to 4½ million gal. a day, but with the growth of the city, and changing industrial conditions, it increased materially until, in 1918, it reached a maximum consumption of 7 072 000 gal. a day, with a maximum day of over 8½ million gal. This increased consumption placed an unduly heavy load on the filter plant designed for a maximum yield of only 6 million gal. a day, and the quality of the water began to deteriorate both bacteriologically and physically.

The bacteriological deterioration was overcome by the introduction of chlorination in 1910, using, at first, calcium hypochlorite. The raw water treated here contains a large amount of paper mill and pulp mill waste, both ground wood and chemical processes being used in the production of the pulp. When we first started the use of the chlorine we fed it into the raw water, but soon met with trouble. The chlorine seemed to act on the sulphur compounds derived from the pulp mill waste in the sludge in the coagulating basins, and before we reached a chlorine feed that gave thoroughly satisfactory bacteriological results we had produced a very disagreeable taste in the water, though at no time was the chlorine feed high enough to give a chlorine test in the filtered water. Changing the feed to the filtered water, which contained much less organic matter and, consequently, required less chlorine, eliminated this trouble. In 1919 a Wallace and Tiernan liquid chlorine machine was installed and the use of hypochlorite discontinued. In 1920 a duplicate liquid chlorine equipment was installed and is always kept in working order for emergency use.

While chlorination took care of the bacteriological troubles very nicely, it did not, of course, remedy the deterioration in the appearance of the water due to color and turbidity. These were due to a short detention period in the coagulation basins, with resulting incomplete coagulation, and to too high rates of filtration. To remedy these troubles, additional filters and increased basin capacity were decided upon. Four new filters of the same type as the original eight were built, increasing our filter capacity by a little more than 50% and allowing us to reduce filter rates to normal again — 120 million gal. per acre per day.

To provide additional coagulation basins, which were our greatest need, use was made of certain natural advantages which we possessed. The city owned most of both banks of the Black River for a considerable distance above the plant, as well as two islands in the river. Before the filter plant was built, the secondary channel around one of these islands had been dammed off and the resulting basin used as a gross settling basin. At a comparatively small expense a concrete dam was built across the upper end of this basin, about a mile upstream from the filter plant, and a chemical feed house with a Venturi meter and concrete mixing-box in-

stalled. The equipment at this station was made large enough to enable us to admit enough water to the basin in eight hours to supply the filters for a full day, with a consequent saving in the cost of operation of this station. This gave us an outdoor coagulation basin of about 50 million gal. capacity and several days' storage, not only allowing the coagulation reaction to come to completion, but also permitting very good sedimentation. There is still enough floc passing through this basin and on to the filters to form a good filtering film. The good effect of this basin is shown by comparing the periods of service of the filters and the amounts of wash water used before and after the basin was put into service. In August, 1917, the filter runs averaged only 5.98 hours and the wash water 548 000 gal. a day, — 7.3% of the total amount filtered, — while in August, 1918, with the new basin in service, the filter runs increased to 15.52 hours and the wash water dropped to 231 000 gal. a day — 2.9% of the amount filtered. It may be noted that this saving of 300 000 gal. of wash water a day was all that enabled us to meet the demands of the city at that time without curtailing service. Increased experience in operating this basin has enabled us to improve these figures materially — e.g., in August, 1923, the period of service was 38.36 hours and the wash water used only 87 000 gal. a day — 1.5% of the total amount filtered. The alum feed and coagulation basins at the filter plant were still retained, of course, enabling us to give the water a secondary dose of alum if needed, which is now seldom the case.

At the same time that the additional filters and new basin were built, a fountain was also built to provide aeration of the water to remove paper-mill tastes and odors, and free carbon dioxide as much as possible, with a decidedly beneficial effect. This aeration takes place after the water has passed through the large river coagulation basin, but before filtration.

These improvements have enabled us to meet the increased demands upon the plant and still maintain a high standard of effluent. In the past few years there has also been a marked decrease in consumption, due to increased metering and more thorough searching for leaks, so that now we have ample capacity for all present needs and increases for years to come.

ROCHESTER'S (NEW YORK) METER-READING AND BILLING SYSTEM.

BY STEPHEN B. STORY.*

[*Read September 30, 1924.*]

There is little or nothing in the field of water-works business which is at all spectacular. The work required to get into the treasury money in compensation for water delivered is certainly as uninteresting to the average person as we can imagine anything to be. However, it is one of the most important functions which water-works men have to perform, for without the wherewithal to pay for materials and labor consumed in supplying water, water-works systems would cease to exist. If effectiveness, simplicity, and convenience to rate-payers are desirable in the money collection work of water departments and companies, a study of methods employed can easily produce results.

It is the purpose of this paper to present to you in a very brief way some of the features of the meter-reading and billing system of the Water Bureau of the City of Rochester.

The city has at present approximately 54 000 consumers who obtain water from the domestic supply. The water used by consumers is entirely metered. There are 1 300 meters on the services of large consumers. Meters on the 52 800 smaller services are read quarterly and on the larger services are read monthly.

It was formerly the practice to read all meters in the city in a group once each quarter. From 41 to 42 working days were required to complete the work. Bills were then prepared and the meter-readers all turned bill-distributors and handed out the bills. Distribution required 11 days. The water bills were for water supplied during the previous quarter.

There is in force in Rochester a system of interest charges which are applied at the rate of one per cent. per month upon all bills which are not paid in the month in which they become due. This acts as an inducement to speed up payment of water bills. The natural inclination of the consumer is to pay his water bill during the last few days of the month in which his bill is due. About two-thirds of the bills are paid in the month they become due. Seventy per cent. of this two-thirds are paid *via* mail, and thirty per cent. are paid by the consumers in person at the office of the city treasurer. The number of bills paid in person is about 11 000. On the last few days in which bills were due it was customary for long lines of people to form while waiting their turn to pay their water bills. Under

* Director, Bureau of Municipal Research, Rochester, N. Y.

the old system, mail from about 20 000 accounts deluged the treasurer's office simultaneously with the rush at the cashier's windows. Work in the treasurer's office other than that pertaining to water revenue collection was at a standstill, and it was usually several weeks before that office recovered from the load imposed upon it by the method of quarterly collections.

On January 1, 1920, the Rochester Water Bureau put into effect a new system for reading its meters and billing its consumers.

The city is divided into three districts of approximately equal size as regards the number of meters in each district. This divides the meters so that there are about 17 000 in each district. A month is required to read a district's meters. In addition to this quota, the monthly readings have to be taken on the monthly accounts. Inasmuch as the ward division is used in many of the descriptive systems in the books of the various city departments, it was felt that ward designations should be retained, and wards are assigned to districts in their entirety. A work program, or schedule for the continuous reading and monthly billing plan, was prepared and put into effect. It has been carefully adhered to and the work has not been permitted to lag behind it.

The meter-reading is done by one group of meter-readers who read meters continuously. As soon as one section is completed on schedule, another section is started. Ten readers are required and 95 readings a day is figured to be a day's quota. This quota means 95 calls. The regular reader gets about 92 per cent. of the readings. A special reader covering the non-read consumers gets 3 per cent. more. The non-read meters are but 5 per cent. of the whole number. Bills are distributed by men who do nothing else. Four distributors are required.

As soon as a day's readings are turned in to the meter-reading division, the read cards are checked and sorted in order of folio, or in order of street and house numbers. As soon as the readings for one ward are accumulated, the billing process is started. The time required to read the largest ward is six and one-half days. After the date of reading, previous reading and present reading and the consumption are stamped on the bills by an adding machine; they are run through a specially designed "pricing" machine which multiplies the consumption by the rate, and stamps the amount upon the bill. Three different-colored papers are used for the three quarterly bills. The bills are then sent to the accounting division where they are checked up and postings made by machine to the account records.

The account ledgers are of loose-leaf type, and carbon copies of entries are made for the comptroller and treasurer. Accounts are entered in order of streets and house numbers, or, in other words, the order of reading meters, as represented by a reader's progress along a street, is the order in which the account entries are made on the ledger pages.

An addressograph is used, wherever possible, in making out read cards, bills, and ledger sheets.

Multigraph equipment enables the bureau to print all its bills and, incidentally, most of the miscellaneous forms for the Department of Public Works.

Forms are used to record all transactions concerning water consumers' accounts. As soon as the necessary entries are made upon the ledgers, the bills are sent to the reading division for re-sorting and distribution. The distribution quota is 300 bills per day.

On the first day of each month a group of water bills representing one-third of the city's consumers is due. The effect of this continuous reading and billing plan is as follows:

On the Meter-Reading divisions.

It eliminates the peak loads which formerly occurred quarterly.

It keeps meter-readers steadily at work doing one thing —reading meters.

It enables better planning and control of work of reading and billing.

On the Accounting Division.

It delivers the bills to the accounting office in a fairly steady stream.

It spreads complaints and high-bill discussions evenly over the entire year.

It enables the work to be schedulized and makes unnecessary the accumulation of bills for a period longer than that required to post the consumers' records and make the necessary proofs.

On the Meter-Repair Division.

Reports of impaired meters are turned in on meter-reading cards. These cards bear notation as to the condition of meters. The quarterly system made peaks in the meter-repair work. By continuous reading repair work comes to the meter-repair division at a uniform rate.

On the Treasurer's Office.

Quarterly peak loads are reduced 66⅔ per cent.

Clerks are employed continuously in the opening of mail and the collection of water revenues.

Better organization is effected in the office force, and other work is possible without interruption.

Water-rent payments are made in the aggregate two months earlier than previously. The interest gained thereby is about $6 000 annually.

Discussion.

W. C. Hawley.* Mr. President, it seems to me that this is a paper which ought to interest every manager present. Mr. Story has pointed out an excellent method of handling the meter-readings and the bills; but the one thing which he has not called especial attention to is the very material saving which a method of this kind effects.

Many years ago, I had a similar problem put up to me, and in that case it was the economy which might result from changes in our methods of reading, billing, and collecting, which was of particular importance. At that time we were reading the meters quarterly, and billing quarterly. The result was a mob of people to pay their bills four times a year, with crowds stringing way out into the street. If one man had a complaint to make, it started the man behind him, and our troubles were multiplied. It took more clerks to handle the business, and as a result of our experience we divided our whole territory into districts. Our main office handles 12 districts. We begin reading the meters in District A, we will say, on the first day of January. Four or 5 days completes District A. Our meter-readers then go to the district which was read three months previous and take what we call an "intermediate" reading, — not a reading from which we make bills, because we changed from the quarterly to the semi-annual basis of collecting. They finish the "intermediate" reading in that district in time to begin at the middle of the month with the reading of District B, and after finishing in District B in 4 or 5 days they go to the district which was read three months previous for another "intermediate" reading.

The result is that our bills are going out every two weeks to one of the districts and the collections are coming in gradually instead of in a bunch. We seldom have more than two or three people in the office at a time to pay bills. We have further reduced the number of people coming to our office by arranging with the banks in the various municipalities which we serve so that they receive payments. That has reduced our work in the office very materially and we have been able to reduce the number of meter-readers. We are reading meters today with fully two-thirds more consumers than we had when we began this system, and we only have two or three more meter-readers than we had at that time.

Recently I have been studying what it might cost us if we had to adopt the monthly system of billing and collecting, and it runs into a good many thousands of dollars a year. It is surprising what a difference it would make.

I think that most of us can make a study of the method which Mr. Story has suggested, and many of us might find that we could adopt something of the kind, not only with benefit in the method of handling the business and avoiding trouble, but with great economy.

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* Superintendent, Pennsylvania Water Company, Wilkinsburg, Penn.

PRESIDENT HEFFERNAN. Let me ask Mr. Story about the meter-readers being able to get into the houses to read the meters. Have you tried the postal card return?

MR. STORY. The occasions on which they find trouble in getting the readings are rare. Any time that a reader cannot get inside we send a postal card notice to the consumer, explaining to him that the reader could not get in and if he will call up the office at a certain time we will arrange to send a man to the house to read the meter.

PRESIDENT HEFFERNAN. You have no card to show the indicator?

MR. STORY. Yes, there is another card which we use. If we cannot obtain a reading we send a card with four or five dials on it so that a person can read his own meter and send it in.

PRESIDENT HEFFERNAN. Have you found that satisfactory?

MR. STORY. I cannot tell you just what result it produces, but I know we use it here.

MR. HAWLEY. I would like to say that we are using a similar card and we are getting a third or a little better of our readings by these cards. And we are getting another result; we are getting people to read meters who never thought of doing such a thing before.

BEEKMAN C. LITTLE.* We send postal cards with the dial on only to those where we cannot get in, and after we have tried the second reading. And we have a good many of them returned in a very satisfactory manner. We very seldom find them wrong. Once in a while they mark them wrong. They do not try to read them. We just tell them to indicate on the dial where the hands are. Once in a while we have to verify the readings.

W. L. HATCH.† I would like to ask a question on this point. We have a great many three-deckers. The usual custom is to put the meters in the front of the house and give the front cellar to the third-floor tenant. The consequence is that our meter-reader has to go up to the third floor to get the key, then return it to the third floor after he has read the meters.

I would like to know if in any places they have an ordinance on the subject, requiring meters to be so placed that they can be more readily read.

PRESIDENT HEFFERNAN. Can any member answer Mr. Hatch's question?

MR. LITTLE. We have in our office, in the meter-readers' section, perhaps 50 to 100 keys that people have left for our meter-readers to get into their houses. They trust us very well. And a great many times they leave the keys next door with the neighbors. We have a list of all these places. It is our experience that in these times a great many houses are closed, — a great many more than there were ten years ago.

As Mr. Story has said, we notify them on a card that we are willing to read one or two nights in the week, Saturday afternoon, or on Sunday.

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* Superintendent, Water Works, Rochester, N. Y.

† Board of Water Commissioners, New Britain, Conn.

We have special readers who will go out at such times, so that we obtain practically all the readings. None are allowed to go over six months.

J. WALTER ACKERMAN.* I would like to ask one or two questions. Relative to this question of not getting the readings, if the reader cannot get into the house and you do not receive any response from the card, do you send them a minimum bill in order to keep the account alive?

MR. LITTLE. No, we do not. We send them a bill at the first quarter that we cannot get in. In that case we send them a minimum bill, which is $1.50. But if we cannot get in and read the second quarter, at the end of six months we notify them that the water is to be shut off, and we shut it off when we have to; but it is very seldom that we have to. The owner or somebody will let us into the house. We only send that one bill, the minimum for the one quarter.

MR. ACKERMAN. What do you do with the people who come to pay the water rent but who do not bring the bill with them?

MR. LITTLE. We look up and see what the reading is on the books and make the bill out in the office.

MR. ACKERMAN. When he comes to the treasurer to pay them, the treasurer of course then pays no attention to the man unless he has the bill with him; or do you have a copy of the reading right there?

MR. LITTLE. The treasurer sends into the accounting office, the next office to his, and gets the bill.

MR. ACKERMAN. The treasurer has something along the line of a commitment sheet to show just exactly what each person is supposed to pay?

MR. LITTLE. No.

MR. ACKERMAN. He has a record in his office of what each one is to pay?

MR. LITTLE. No.

MR. ACKERMAN. The only record, then, that he has is the bill the person hands in?

MR. LITTLE. Yes. And that is checked up at the end with our account, — with the water meter account.

MR. HAWLEY. We insist upon getting the reading. We send a man a second time and, if necessary, we leave the card. If we do not get the reading then, we send a return postal saying that our man has been there on a certain date, has left the card, and we must get the reading of the meter; and unless they arrange it so that we can get the reading, we shall assume that they want the supply discontinued. And we get the reading.

FRANK E. WINSOR.† I did not come prepared to discuss this paper, but feel that the methods used in collection of revenue in Providence, R. I., the system being almost entirely metered, are of interest. I regret that the Commissioner of Public Works, Milton H. Bronsdon, a member of this

* City Manager, Watertown, N. Y.
† Chief Engineer, Water Supply Board, Providence, R. I.

Association, who is in charge of the present Water Works, is not here to discuss this paper, as his experience in the collection of revenue would undoubtedly bring out important features which I may overlook.

Up to March 1, 1923, the practice was to consider the city as one unit, all bills being collected annually with the minimum charge collected in advance, except in case of a relatively few large consumers who were billed quarterly, with a still smaller number billed monthly.

Beginning with March 1, 1923, the city was divided into four approximately equal parts, the yearly bills for each quarter being collected, respectively, in March, June, September, and December, thus spreading the bookkeeping throughout the year. Otherwise, the former practice as to payment, annually, quarterly, or monthly still prevails. All meters are read quarterly, but only those meters indicating a large consumption in the previous year are billed oftener than annually. The minimum charge per tap is $8 annually, and about 15 per cent. of taps come in this class. The per cent. of taps paying annual bills is 95, quarterly bills 4, and monthly bills less than 1 per cent. As compared with the collection of all bills quarterly, a great amount of bookkeeping and other expense is avoided and the results are entirely satisfactory. The water works serve a population of about 300 000 and there are about 38 000 taps.

STEPHEN H. TAYLOR.* In New Bedford we stick to the method which has been referred to this morning as being old-fashioned. We bill quarterly, although we read monthly for the purpose of checking up leaks, stopped meters, and so forth. We have about 18 000 accounts. In our case the minimum charge is payable yearly, not quarterly, so that our big billing is bound to come once a year unless we can make the minimum charge quarterly instead of yearly. I suppose that not more than half of the accounts will run over the minimum charge. There may be some advantage in this monthly billing, making the distribution of work more even, and I was thinking as I sat here that I would take the question up with our water registrar when I got back, to see if the scheme can be worked out with our system.

We do not try to get a house-owner to read his meter. We look to the owner of the property for payment and not to the tenant. In many cases the owner of the property lives a great distance away from the property. For instance, in a house that has six or eight tenants, whether we would be able to get one of them to send a card with a correct meter reading on it I think would be quite a question. We have no difficulty, though, in picking up on the second visit, arranging by telephone or otherwise for a time when we can get in. As in Mr. Little's case, we have quite a collection of cellar-door keys in our office, and sometimes we can get the keys at the next door. We have no particular difficulty in getting our readings.

GEORGE F. MERRILL.† I think there is one advantage to the monthly

* Superintendent, Water Works, New Bedford, Mass.
†Greenfield, Mass.

readings, and that is that it gives the meter department a chance to check up the meters that are leaking. A certain percentage of meters go dead every little while for one cause or another and therefore cause a loss of revenue to the department, particularly on the large meters. It is also a good thing to give the consumers, where they have leaks, a chance to protect themselves.

LARGE AND SMALL FIRE HOSE, WATER PRESSURES, AND HYDRANT SPACINGS.

BY WILLIAM R. CONARD.*

[*Read October 3, 1924.*]

Recently there has been much written and spoken, both from water and fire department sources, on these three subjects:

Fire Hose, large and small.

Water Pressures and their relation to fire fighting.

Fire Hydrants and the real economy in their proper spacing.

While these have been live issues for many years, there has never been as hearty a coöperation among fire and water departments and fire and water engineers as is now the case.

While it is intended to confine this brief paper to the subject of Large and Small Fire Hose, the matter of water pressures and hydrant spacing are so closely identified with the question of the size of the hose that some reference will, necessarily, be made to the other two.

The size of the hose most generally used for fire-fighting purposes, either attached directly to the hydrant or to pressure-boosting apparatus, is 2½ in. which at 90 lb. pressure at the hydrant gives a flow of approximately 195 gal. per minute through 500 ft. of hose, with a pressure at the hose nozzle of approximately 20 lbs. This size hose has been in use so long that it is looked upon as *the* standard size, and other sizes, while in reality standards, are looked upon by many as odd sizes. The reason for this is that for sizes smaller than 2½ in., much of the hose is not considered substantial enough, and the friction losses out of proportion to the usefulness for those fires that reach anything like serious proportions. Also, on the other hand, many consider that 3-in. hose is too awkward to handle to be really useful, on account of the number of men required, and the heavy work it places upon them. This is true to an extent, but the working pressure at the hydrant nozzle, and amount of hose necessary to use, are also very important factors in considering whether hose is too small or too large; and this is particularly true for the 3-in. size when used in connection with very high pressures and in attempting to use long lines of that size.

Without attempting to detail the reasons for the conclusions of the writer, it is his judgment that except in localities having tall buildings and covering considerable ground areas, a water system so constructed as to supply an ample quantity of water for a sufficient number of hose streams to handle the ordinary fire at a pressure of 90 lbs. with the hydrants spaced

* Consulting Engineer, Burlington, N. J.

not over 200 ft. apart, is what the ordinary community should have. Such a pressure would not prove a burden on the plumbing in the buildings and yet would give sufficient water to the shorter lines of hose (which the short spacing of hydrants would effect) to take care of a fire on the upper floors of the ordinary building up to even 6 stories, and at the same time would in most cases enable the firemen to work at a safe and comfortable distance from the building on fire. This would not mean the elimination of pressure-boosting apparatus, as the average community is usually subjected at some time in its life to a fire that reaches serious proportions, entailing an excessive draft on its water system, and necessitating means of getting higher pressure through longer lines of hose; in fact, there have been instances where, if there had not been pumping apparatus, relaying from one pump to another to get water to a distant point, a conflagration with loss of life and high property damage would have resulted.

Granting that 2½-in. appears the best size of hose to use, let us see what results we would get by using a smaller size and a larger size, alone and in combination.

Take 3-in. hose, and assume that we have the hydrants at short intervals. It would be possible through 6 lengths, or 300 ft. laid from the hydrant, to obtain 335 gal. of water at 90 lbs. pressure at the hydrant nozzle, as against 235 gal. if the hose were 2½ in. This larger hose would be placed before the water had been turned on, and would not likely be disturbed until the "all out" signal had been given; and empty 3-in. hose is practically as easy to handle as 2½-in., yet 100 gal. per minute more water would be supplied.

One and one-half (1½) in. hose, 200 ft. long, will supply 95 gal. of water at 50 lbs. pressure, which would be the pressure at the nozzle end of 300 ft. of 3-in. hose with 90 lbs. at the hydrant; or within 100 gal. of 500 ft. of 2½-in., with 90 lbs. at the hydrant. Therefore it would be possible to siamese the 3-in. to two 1½-in. hose lines, either of which could be handled by one man as against two for the 2½-in., and besides getting within 15 gal. of as much water, be able to attack the fire from two points instead of one.

In former days, when hose-carrying apparatus was propelled either by hand or by horses, extra weight made a very material difference as between getting to the fire quickly or being delayed and tired out, whether of humans or animals; where today, with modern motor-propelled apparatus, weight does not count so much as the time factor, which is the all-important factor in the early stages of any fire.

If the fire appears to be assuming proportions that necessitate the use of 2½-in. hose, there would still be an advantage in having your 3-in. hose laid for, say, 300 ft., and by taking off your 1½-in. hose and hooking the 2½-in. hose onto the 3-in. and using single lines. This would give you 75 gal. more water than if the fireman was depending on the 2½-in. from the hydrant to the hose nozzle.

By this you will see that I am not advocating the discarding of the

2½-in. hose, but rather believe that: first get the spacing of the hydrants at a maximum of 200 ft.; then arrange the hose-carrying apparatus of the fire department to carry 300 ft. of 3-in. hose, 400 ft. of 1½-in. hose, with 500 ft. of 2½-in. hose, together with nozzles for 3-in., 2½-in., and 1½-in., and the necessary adaptors for working the various combinations; and if your pressure is around the 90 lbs. mentioned, you can cope with all except the most serious fires, and, at the same time, avoid the necessity for any considerable amount of pumping apparatus. There is the further advantage in keeping the other apparatus, which is lighter and more mobile, at high efficiency and in ample numbers; and also, in the majority of cases the fire will be controlled as quickly, if not more so, and with very much less water damage.

In a manner confirming the foregoing conclusions of proper methods, I would quote a few extracts from other papers on the subject:

"Fire and Water Engineering," October 24, 1923, by F. L. Bock, Engineer.

"The type of fire hydrant usual to small cities and towns has two nozzles or outlets, each for a 2½ in. hose coupling. Such hydrants are generally intended to supply hose lines direct from the hydrants, and the pressure desirable in such systems is 75 to 100 lbs. if effective fire streams are to be obtained. At least, the pressures in the water mains when delivering substantial fire flows should not drop below 50 lbs. where there are only two-story buildings; not less than 60 lbs. with three-story buildings, and not less than 75 lbs. where there are considerable four-story buildings. In fact, more or less fire-engine capacity is generally advisable, or necessary, where four-story or higher buildings prevail. Hydrants with double 2½-in. outlets would normally be expected to deliver 500 gal. per minute, without appreciable pressure loss, if connected with 6-in. pipe, but the loss becomes considerable when 4-in. connecting pipe to water main is used.

"The distribution or number of hydrants for a given area should depend principally upon the fire flow required to properly protect a district, which in turn depends upon the size, density, and combustibility of the district. Where high pressures are provided, as from fire engines, there should generally be one for each 40 000 to 120 000 sq. ft. of ground area, and where direct hydrant streams are used, one for each 40 000 to 100 000 sq. ft.; in either case depending upon the fire flow required.

"Let us consider the proposal to use 3-in. fire hose for all outside streams, and to supply 2½-in. hose lines within buildings. The first important consideration in this connection is the coupling of 2½ with 3-in. hose. To eliminate all possible trouble in this respect, the 3-in. hose should, undoubtedly, be equipped with beveled 2½-in. couplings. With this arrangement there is practically no interference with the flow so long as the apron of the coupling is properly tapered; which means nothing more nor less than a slight nozzling and reverse nozzling action. As we know, good nozzling action does not cause a perceptible loss of pressure but converts one kind of pressure into another. Therefore, 3-in. hose can be coupled indiscriminately with 2½-in. hose."

"Fire and Water Engineering," November 21, 1924, by E. H. Barrier, Assistant Chief Engineer.

"In view of the great advantage of the 3-in. hose, it is hard to see why fire departments on the larger cities have been so slow to adopt it. Its use is increasing, and this largely due to the praiseworthy campaign which the National Board of Fire Under-

writers has conducted, but there is still a large number of cities where 3-in. hose could be adopted to advantage.

"Two and one-half inch unlined linen hose, which may be considered to have about the roughest interior of any hose in use, showed a drop in pressure of over 32 lbs. per 100 ft., with 250 gal. per minute flowing. Two and one-half inch cotton rubber-lined hose, with a badly corrugated interior, showed 25½ lbs., while good cotton rubber-lined hose showed but 14 lbs. The importance of this can be quickly appreciated if we consider a 500-ft. line of hose. In order to deliver an average good hose stream of 250 gal. per minute, which with a 1⅛-in. nozzle requires 45 lbs. at the nozzle, a pressure of 205 lbs. will be required at the hydrant or pumper with linen hose, 162 lbs. with poor cotton rubber-lined hose, and but 115 lbs. with smooth cotton rubber-lined hose. No further argument is needed to emphasize the importance of a smooth waterway.

"It is appreciated, of course, that the 3-in. hose is in the nature of heavy artillery, and there is a legitimate use for smaller hose in the smaller fire departments, or for part of the equipment in the larger departments. In this connection, it is of interest to note to what an appreciable extent in increase in diameter, even as small as ⅛ in., will decrease the friction loss. Taking again a 500-ft. line with 1½-in. nozzle and 60 lbs. pressure at the nozzle, with 2½-in. hose, 211 lbs. pressure is required at the hydrant or pump, while with 2⅝-in. hose approximately 175 lbs. would be necessary — a gain of 36 lbs.

"The use of 2⅝-in. hose in place of 2½-in. has been quite general in private factory equipments, but for some reason or other fire departments have adhered to the smaller hose. This is probably due to the fact that the matter has never been properly presented to them. The 2⅝-in. hose has much in its favor and nothing of importance against it."

"Fire and Water Engineering," December 12, 1923, by H. L. Stanton, Fire Chief.

"The difference of friction between 2½-in. and 1½-in. hose is considerable, and some may protest on that score alone; but the advantage is still with 1½-in. hose under most conditions. The fire service of this country is, at this time, responding to alarms of fire so quickly with motor apparatus over horse-drawn apparatus, that seventy-five per cent. of all fires that are not held with chemical lines or hand chemical extinguishers, can be extinguished with 1½-in. hose with a ½ or ⅝-in. nozzle tip, thus saving much property from water damage.

"It is not the intent of the writer to advocate 1½-in. hose for fires that require 3-in. pumping lines reduced to 2½-in. size in order to handle, but to try and present some of the advantages of 1½-in. hose.

"Many companies respond to fires with four men or less, and with a chemical line working on the inside. With one man handling the streams, the balance of the crew can handle the 1½-in. water line with ease as compared with handling a 2½-in. line. Most fires occur in dwellings and the loss from water, in many cases, exceeds that of the fire on account of the use of 2½-in. hose with 1⅛-in. or 1¼-in. nozzle tips. It is conceded that to get the best results from its use, a quick alarm is necessary, whether it is over the fire-alarm system, telephone, or otherwise.

"Many departments first used 1½-in. hose for wetting down fires, or, as many in the service call it, chasing sparks, but it was soon evident with a quick response that 1½-in. hose would handle the entire job. The writer does not advocate all lines of hose laid at a fire to be of 1½-in. size, but with three companies responding on a first alarm one line should be 1½-in. size, with the expectation that it would handle any interior fire following a chemical line. If not, lines of 2½-in. hose from the other companies responding would be available.

"If the fire service of the country is to assist in trying to reduce the water damage at dwelling-house fires, I am of the opinion that the use of 1½-in. hose with 2½-in. tips

will accomplish the purpose. It is not the volume of water required to handle most fires, but rather the getting water in the right place, and getting it there in quick time."

"Fire and Water Engineering," December 12, 1923, by S. H. Short, Fire Chief.

"Prior to 1915, 1½-in. hose was used only in cleaning up after a fire. Very little thought had been given to bringing it into general use as aid in extinguishing fires. In the latter part of 1915, upon my recommendation, each of the companies comprising the Oakland Fire Department was equipped with 400 ft. of 1½-in. hose, a divider (2½-in. to 1½-in.), and two shut-off nozzles, each having two tips, ¼-in. and ½-in.

"The equipment, made up and carried on the apparatus in a separate compartment, consists of a short 3-in. lead line, the Gorter nozzle shut-off attached to same, with barrel removed, to which is attached the divider with 100 ft. of 1½-in. hose connected to each outlet, and to this is attached the nozzles with ¾-in. tips. It is so made up that on stretching in a line, all that is necessary is to break the coupling at the wagon, join on the lead line, and then lead in the two lines where necessary. This leaves 200 ft. for a change of hose.

"Obviously, to extinguish a fire we must reach its base. This is done more easily with the small line. For example: We have a fire in the basement of a four-story building having six vertical unprotected openings, such as elevator shafts, stairways, hoists, pipe chases. Using the large hose, these openings would require a line each with at least twelve and, possibly, eighteen men. These same openings could be amply protected with three large lines equipped with 1½-in. lines, using six men to handle them, reducing your hose, number of men, and pumps by half."

"Fire and Water Engineering," January 16, 1924, by C. H. Fox, Member American Society Mechanical Engineers.

"Approaching the topic, namely, 'The Use of 3-in. Hose,' there needs to be emphasized initially, that an analysis which is to afford satisfactory deductions, must necessarily take into account all of the various factors concerned with the production of effective fire streams.

"Aside from the hose, quite a number of other things enter for consideration and at the outset, therefore, we shall also have to concede that the hose used by firemen — regardless as to its kind — constitutes but one link in a chain of other equally important essentials and which, collectively, serve to create the so-called fire stream. Customarily designated and referred to in terms expressing the nominal inside diameter in inches, any sort of fire hose is to be regarded as a form of flexible conduit designed for the purpose of conveying water, and as to the various sizes in common use, the greater footage, by far, is undoubtedly in the kind known as 'two and one-half inch hose.'"

"Proceedings American Water Works Association," January 2, 1924, pages 209, 211 and 212.

"The mistake, as I see it, has been made in many places of trying to get along with a few hydrants by using long lines of hose. That is very expensive, and the friction loss is so great that much of the pressure is lost before it reaches the nozzle.

"The city should be provided with pumpers to take care of any additional fire pressure that may be necessary. In a talk with one of the officials of the Board of Fire Underwriters he expressed the opinion that, in a city of 50 000 or larger, pumpers or fire engines should be provided to take care of the pipes.

"I agree with the last speaker that a good pressure is certainly to be desired in a water-works distribution system, but there are limits beyond which it is unreasonable and unnecessary to go. I should say an average pressure of about 70 lbs. is to be considered excellent and, generally, satisfactory for all usual purposes other than fire protection for the higher buildings.

"When pumping apparatus is used, the question of whether the pressure of the hydrant is 50 or 100 lbs. is, I believe, of no moment. Up to a certain point the pressure of the hydrant, of course, reduces the amount of work to be done by the pumper. But the pumper should be of adequate capacity to deliver its quota of satisfactory fire streams under all pressure conditions, so that it is the job of the water-works man to see that there is sufficient pressure available to deliver the required amount of water through the hydrant to the pump. Whether this pressure is 10 or 100 lbs. is of no consequence, as long as the water, in sufficient amount, is there."

SANDSPUN PIPE.

BY CHARLES R. WOOD.*

[*Read October 1, 1924.*]

Without having made a very thorough study of the actual mechanics of the new process of cast-iron pipe manufacture, I will endeavor to give you a brief outline of the things which have been accomplished during the last year at the experimental plant operated by ourselves and our associates at the works of the American Cast Iron Pipe Company at their plant in Birmingham, looking toward the development of spinning cast-iron pipe in a sand mould.

To describe the process briefly: We take a cast-iron mould and jolt ram it with green sand in a vertical position. This mould, or flask, is then placed in a horizontal position upon rollers and revolved at a fairly high rate of speed, after which molten iron from an ordinary cupola and by means of an ordinary tip ladle is poured into the revolving mould. When sufficient metal has been placed therein, the speed of spinning is very materially increased for a couple of minutes, after which it is slowed down, the metal cooling and the mould removed from the rollers, when the pipe is pushed out of the sand on to the skids. This, in a few words, describes the process as it is now in operation.

Our people — and by that I mean those of us who are interested in this Birmingham development — feel that castings made in this way will prove of merit because of the fact that we have avoided any chill in manufacture and because pipe made as described above has turned out to be of the best texture that we have ever had in our shop. While being fairly tough, it has been very easy to machine, and certainly the threads and holes tapped in the pipe which we have ourselves machined in our own shop at Florence, N. J., show most excellent metal, — as many of you will undoubtedly have noticed from the samples which we have distributed in various parts of New England and from those which are now on exhibit in the other room.

The plant at Birmingham was developed mainly under the supervision of W. D. Moore of the American Cast Iron Pipe Company, who is largely responsible for the thoughts and ideas which have gone into the manufacture of the pipe as you see it today; and the writer had hoped that it might be possible for Mr. Moore to write something about it for you. But since the death of Mr. Egan, a few months ago, Mr. Moore has been made president of the American Company and has been unable to give the time needed to prepare even a brief outline of the work that has been done.

In the experimental plant, which has now been operating for about a

* Of R. D. Wood & Co., Philadelphia, Penn.

year, only 4-in. and 6-in. pipe have been made for commercial uses. In the near future, the plant will be prepared to turn out 8, 10, and 12-in. pipe and before very long we expect to be manufacturing in sizes up to 24-in. in diameter.

The pipe we have made so far has been very satisfactory, although it has occasionally a somewhat rough-appearing interior surface, caused by oxidation when cooling.

Perhaps I should have stated that we are covering the green sand moulds prior to casting with practically neat cement, so that the outside of the pipe is coated with cement, and we have also been lining the pipe inside with cement, performing this latter operation by spinning on rollers in the yard after the castings have been cooled and cleaned. Parenthetically, it might not be amiss for me to suggest that this method of lining the inside of the pipe has been rather enthusiastically endorsed by a paper recently read before you, although I fail to remember having heard anything said about its particular advantage when used with cast-iron pipe.

It seems, after what we have heard recently, that it will not be necessary for me to go into many details about the advantages of cement lining, and we believe it is now almost unanimously considered that cast-iron pipe, properly lined with a good cement, is what the world has been waiting for, and we believe that in this sandspun pipe, with its good, clean metal, free of impurities, well and thoroughly cement-lined, we will be able to furnish a conduit that will greatly reduce friction loss and last — forever.

This sandspun pipe is a new product, and when lined with cement is a little different from what users have been handling and, therefore, must be laid with a little more care than ordinary sand-cast pipe has been in the past. By this I mean that where pipe is cement-lined, it is necessary that a little more care be taken when drilling and cutting pipe in field work.

We, ourselves, have done some very satisfactory work in cutting this pipe in our own yard with a three-wheel cutter, there being no sign of cracks in the lining.

So far, only very tentative specifications have been prepared for sandspun pipe, but we expect to be able to reduce the present class weights, so that pipe usually made to Class "B" dimensions can be spun of Class "A" thickness and be amply strong for service for which Class "B" sand-cast pipe is now used.

We have tested full-length sandspun 6-in. pipe, 0.44-in. thick, up to 2 000 lbs. per sq. in., and 4-in. pipe, 0.42-in. thick up to 3 500 lbs. before breaking, these tests being made under hydraulic pressure.

If the increase in frictional loss which now occurs with the aging of cast-iron pipe is entirely obviated, as we expect it to be, we believe that for work in which you have been accustomed to use, say, Class "C" 10-in. pipe, your engineers will find that it will be perfectly safe to use Class "B" 8-in. pipe, as it will be unnecessary to design your water line, looking forward to the expectation of decreased capacity on account of tuberculation.

The manufacturers who have been interested in this new development at the Birmingham experimental plant are: The American Cast Iron Pipe Co., of Birmingham, Ala.; Donaldson Iron Co., at Emus, Penn.; The Glamorgan & The Lynchburg Companies at Lynchburg, Va.; Warren Foundry and Pipe Co. at Phillipsburg, N. J., and R. D. Wood & Co. at Florence, N. J.; all of whom are prepared at the present time to deliver sandspun cast-iron pipe either with or without cement lining in 4-in., 6-in. and 8-in. sizes and will be prepared to furnish larger diameters in the near future.

While, as he said before, the writer is not entirely familiar with all the mechanical details of this new operation, he has tried to give you briefly a description of what has been accomplished at the Alabama experimental shop during the past year, and will be glad, in so far as it is possible, to answer any questions that you care to ask in connection with the work which has been done.

In conclusion, I want to explain to you that Walter Wood, who had hoped to present his thoughts to you on these matters, has been unable to attend this meeting, and the writer has, at his request, prepared this short paper, somewhat hurriedly, in order that we might not disappoint your programme committee in their very courteous request to have us give you a short description of this new sandspun process.

Discussion.

President Heffernan. Is there any special method which you use for drilling through the cement in making service connections?

Mr. Wood. We have been experimenting with very careful drilling in our shop. We do not think there is going to be any trouble. We have sold a large amount of pipe in New England and in our part of the country, and we have not had any reports of trouble. Of course it is a little hard to get inside of a 4-in. or 6-in. pipe, but I do not think that the people who have been using cement pipe in New England have ever had any particular trouble with compacting the cement down inside.

President Heffernan. I did not know but you were using a special tool.

Mr. Wood. We have not got that far yet.

Caleb M. Saville.* I hold no brief for any of the pipe-manufacturers, and I have not any stock in any of their concerns, but at the American Water Works Association Convention, this spring, Mr. Wood told me about this pipe and we bought some of it and we put it in the ground. Before putting it in the ground I told the pipe-laying foreman to go out and cut off some ends and see if he could crack the lining off, and the service-pipe foreman I told to go out and tap it in his ordinary way and see what happened to the inside of the pipe. They both reported that they cut it

* Manager and Chief Engineer, Board of Water Commissioners, Hartford, Conn.

off very nicely without any trouble, with the ordinary chisels, and the service-pipe foreman has told me several times since that he hoped we would get some more of that pipe.

CHARLES W. SHERMAN.* I do not know whether Mr. Wood is prepared to answer it, but the question arises in my mind whether there would be any difference in the cost of this sandspun pipe as compared with the ordinary sand-cast pipe; and if so, whether it would be more or less.

MR. WOOD. As far as the sandspun pipe is concerned, I think it will be less. I am a little skeptical about what the costs of cement lining are going to be, because when I inquired about the matter I found they were higher than I thought they would be. I then decided to find out what tar coating was costing, and that is very expensive, too. We are going to be able to furnish a lighter pipe, and I think we have been selling it at five or six cents lower per foot than we have the ordinary sand-cast pipe. You must understand that this pipe is being made in the experimental plant in small quantities. When the additional plants are built we expect a very much cheaper price. Whether it will result that way or not I do not know.

MR. SHERMAN. If I understand Mr. Wood correctly, we have a reasonable chance of obtaining this pipe at a somewhat less cost than the ordinary pipe, principally because we can use a lighter weight of pipe. Is that it?

MR. WOOD. Yes.

WILLIAM R. CONARD.† As I understand it, the cost of coating is a question largely of the cost of the operation of the mechanical devices used to coat the pipe,—I am speaking of coal tar now — the type of oven, the cost of the fuel used in the oven for re-heating the pipe previous to the coating, and the amount of labor necessary to be used in the coating process, —and not the cost of the coating itself. And by the same token, of course the cost of cement lining will resolve itself down to, not so much the cost of the cement and the mixing of the cement itself, but the cost of the labor and the apparatus for applying it.

SECRETARY GIFFORD. Did Mr. Wood say anything about the liability of damage from handling the cement pipe; if so, will he repeat it for my benefit?

MR. WOOD. We have not heard of any trouble with transportation. I had thought that in cutting the pipe in the field it would probably be necessary to take a little more care than is sometimes done, especially by contractors; that is, I thought that there might be danger of the cement breaking off. But as Mr. Saville has kindly said, they had no trouble with the pipe they bought.

SECRETARY GIFFORD. Last week, before coming here, I had a carload of the cement-lined pipe, and I cautioned my foreman that it was coming and to be very careful of it; that I was afraid the cement lining would break. He said, "Oh, it is all right; it is all off the car, and I dropped a

* Of Metcalf & Eddy, Boston, Mass.
† Consulting Engineer, Burlington, N. J.

dozen lengths on the ground to see if it would break. It is all right."

MR. SAVILLE. Just to show that I am entirely impartial in this matter I want to say that we have used the centrifugal pipe also, and that we tapped that very nicely.

PRESIDENT HEFFERNAN. Mr. Saville probably used a little more precaution than our Secretary in unloading his pipe.

MR. SHERMAN. Just one more word: I do not know as it adds anything to the discussion. It seems to me, in the centrifugal pipe and this sandspun pipe, the developments of the last few years have shown us a very remarkable advance in cast-iron pipe manufacture.

ROBERT C. WHEELER.* Do I understand Mr. Wood to say that if the cement lining is adopted, that the diameter of the pipe is less by the thickness of the cement?

MR. WOOD. That is what we are doing at the present time, but we are going to make what the people want.

MR. WHEELER. Some uniform system should be adopted if the cement lining is going to be largely used, so that our tables for flows, etc., will not have to be always corrected by some fractional revision for a difference in diameter. If we are going to have some set thickness of lining or have some uniform size, it would be a good plan to decide on that before the commercial practice has started in two or three different ways.

FRANK A. BARBOUR.† I would like to ask whether it is necessary to replace the sand in the mould at each operation?

MR. WOOD. It certainly is; yes, sir.

MR. BARBOUR. The sand is entirely demolished?

MR. WOOD. Yes. It is run through the sand mixer and put back in the next mould.

---

* Of Barker & Wheeler, Albany, N. Y.
† Of Barbour & Dixon, Boston, Mass.

THE CORROSION PROBLEM IN CONNECTION WITH WATER WORKS ENGINEERING.

BY F. N. SPELLER.*

Wastage Due to Corrosion.

An attempt has been made to estimate the loss due to corrosion. The figures naturally vary quite widely. The writer has been able to obtain some actual data on the amount of steel pipe and sheets used for replacement purposes due to corrosion. From these and similar data from other sources, it appears that in the United States the steel replaced every year for this cause amounts to between one and two per cent. of that in use, i.e., between four and eight million tons per annum. Not long ago, H. S. Christopher presented some interesting statistics before the Pacific Coast Gas Association, which indicated that four million dollars a year was being spent in maintaining gas pipe in California which cost in the first place fifty million dollars.† One gas company alone estimated that if they could prolong the life of their pipe underground one year, it would mean a saving of $25 000 a year in replacement charges. Fortunately, soil corrosion and water corrosion rarely occur at their worst in the same locality. In New England, water corrosion is much more important than corrosion underground, whereas in California, soil corrosion is about fifty times as active as in New England.

Cause and Theory.

Certain facts have been established regarding corrosion. In the first place, it is well known that this cannot occur in the absence of either oxygen or moisture. The effect of oxygen in solution is greatly influenced by the temperature, rate of motion, formation of protective coatings, concentration of dissolved salts, acidity or alkalinity, and many other minor factors. Everyone knows that corrosion does not usually occur uniformly over the surface of metal. Sometimes a pipe will show only a few deep pits. It has usually been observed that the fewer the number of pits, the deeper they are. The localization of this action is mainly affected by contact between dissimilar materials, variations in the concentration of solutions in contact with the metal, particularly where differences in the oxygen contents occur. Other factors affecting the pitting action are the conductivity of the solution, the presence of strain in the metal, the relative size of positive anodic areas, stray current electrolysis, and turbulent flow.

The action of solutions carrying different amounts of oxygen is particu-

*Metallurgical Engineer, National Tube Co., Pittsburgh, Penn.

†*Gas Age Record*, pp. 633–8, Nov. 1, 1924.

larly significant, as it explains the tendency for corrosion to continue on certain areas until the metal is perforated. It has been shown recently that areas in contact with solutions rich in oxygen are cathodic to areas which are relatively poor in oxygen; i.e., the metal will tend to go into solution more rapidly at those places which are shielded and protected from oxygen. This is contrary to what might be supposed to be the case at first thought, but is entirely in accord with the Electrochemical Theory of Corrosion, which is now generally accepted in explanation of corrosion phenomenon, as corrosion in water proceeds by the removal of the hydrogen on the cathodic surface by oxidation. The presence of mill scale and other cathodic materials tends to intensify this pitting effect. It has also been proved that the actual amount of corrosion going on over a unit of area is nearly the same, whether all the surface is exposed to the water, or only a portion. Therefore, we naturally find that the smaller the area exposed, the deeper the pitting.

At one time the composition of the metal was supposed to be the major factor. Ten or twelve years ago the author had the pleasure of taking part in a discussion at a meeting of your Association with his respected friend, the late George Schumann, who believed that wrought iron was inherently much more resistant than steel. Subsequent observation and experience by a committee of the American Society for Testing Materials, and others, have shown that this is not the case. The reason for the opinion held by some seems to be based on their observations of atmospheric corrosion to which roofing material and wire are exposed. In some cases a marked difference was found under such conditions, which has been shown to be due to the copper contents of the metal. Nearly all the wrought iron made in the early days in this country carried considerable amounts of copper. It has been shown more recently by practical tests in service that copper confers on the metal much better lasting qualities under atmospheric corrosion. It has also been demonstrated, however, that, as a rule, copper-bearing iron or steel does not last any longer under water or underground than ordinary steel or iron; as a matter of fact, under these conditions, the composition of the metal has very little, if any, influence within the limits of variation found in commercial material.

In this connection it is interesting to note that while sulphur has a deleterious influence under atmospheric corrosion, the presence of as little as .15% copper neutralizes the influence of 0.10% sulphur. It now appears that copper-bearing steel has a legitimate field wherever the metal is exposed to the atmosphere in the form of wire, roofing sheets, rails, tie plates, steel cars, etc. But, as a rule, experience does not warrant the additional expense of this material for use underground or under water. In the latter cases, external factors control the life of the material. It is evident, therefore, that the confusion in opinions on this subject in the past has been largely due to the lack of a sufficiently broad view of the subject, and to the application of certain observed facts obtained under one set of

TABLE 1. SHOWING RESULTS OF HOT WATER

Time Removed	Locality.	Length of Time Pipe Was in Service.	Character of Service.	Number of Pieces.
1915	Pennsylvania Bldg., Philadelphia, Pa.	2 yr. 1 mo.	Hot water return line.	Eight wrought iron and eight steel pieces connected alternately in a coil.
1916	Irene Kaufmann Settlement, Pittsburgh, Pa.	10 months.	Hot water system.	Four pieces of wrought iron and seven pieces of steel.
1917	Test conducted in three New York City bath houses— (1) West 41st St. (2) East 76th St. (3) East 109th St.	(1) 2 yr. 9½ mo. (2) 2 yr. 5 mo. (3) 2 yr. 6 mo.	Hot water system.	(1) Three pieces each of wrought iron and steel. (2) & (3) Two pieces of wrought iron and four pieces of steel.
1918	Brown University, Providence, R. I.	11 months.	Hot water system.	Two pieces each of black wrought iron and steel; one piece of galvanized steel.
1919	Harvard University, Cambridge, Mass.	3 years.	Hot water system.	Two pieces each of wrought iron, "Scale-free" butt weld steel; and galvanized steel.
1919	Irene Kaufmann Settlement, Pittsburgh, Pa.	2 years.	Hot water system.	Four pieces each of wrought iron and steel.
	Additional Tests in New York City bath houses—			
1919	(1) Cherry and Oliver Sts.	2 yr. 5 mo.	Hot water service.	(1) 1 wrought iron, 4 steel.
1921	(2) Allen St.	5 yr.		(2) 2 wrought iron, 4 steel.
1921	(3) West 41st St.	4 yr. 6 mo.		(3) 5 wrought iron, 15 steel.
1921	(4) West 41st St.	10 yr.		(4) 1 wrought iron, 5 steel.
1921	(5) East 76th St.	4 yr. 7 mo.		(5) 2 wrought iron, 4 steel.
1921	(6) Rutgers Place.	5 yr. 8 mo.		(6) 3 wrought iron, 2 steel.
1924	(7) 109th St.	7 yr. 6 mo.		(7) 2 wrought iron, 4 steel.
1921	William Penn Hotel, Pittsburgh, Pa.	4 yr. 7 mo.	Hot water system.	Four pieces each of wrought iron and steel.
1923	Widener Bldg., Philadelphia, Pa.	6 yr. 8 mo.	Hot water system.	Three lengths each of wrought iron and steel, and four lengths of "Scale-free" butt weld steel pipe, connected alternately.

* These values include only the deepest pit in
† Two inch (2") pipe used in all tests except

SERVICE TESTS. CONDUCTED BETWEEN 1911 AND 1920.

Authority and References.	AVERAGE OF DEEPEST PITS IN INCHES.		MAXIMUM PENETRATION IN INCHES PER YEAR.	
	Wrought Iron.	Steel.	Wrought Iron.	Steel.
Mr. Munsey, Engr. of Pennsylvania Bldg.	.089	.086	.0427	.0413
J. O. Handy, Mgr. Research Dept., Pgh. Testing Lab. *Trans. Am. Soc. Heating & Ventilating Eng.*, p. 125 (1917).	.125	.126	.150	.151
J. S. Macgregor, Inst. in Civil Eng., Columbia University.	(1) .073 (2) .085 (3) .077	.064 .088 .043	.0261 .0352 .0313	.0228 .0364 .0172
W. F. Kenerson, Prof. of Mechanical Engineering, Brown University.	.080	Blk. .074 Gal. .068	.0872	Blk. .0807 Gal. .0742
M. C. Whipple, Inst. in San. Chem., Harvard University. — JOUR. N. E. W. W. ASS'N, p. 42, March, 1920.	.073	"Scale-free" butt weld— .045 Gal. .078	.0244	"Scale-free" butt weld— .015 Gal. .026
J. O. Handy, Mgr. Research Dept., Pgh. Testing Lab. Report made to Resident Director Teller, Jan. 22, 1920. — JOUR. A. S. H. & V. E., p. 207, March, 1920.	.114	.110	.057	.055
Prof. Charles Baskerville, Prof. H. P. Coats, College of the City of New York.	.082 .062 .059 .089 .098 .065 .059	.089 .065 .058 .092 .057 .087 .048	.034 .012 .013 .009 .021 .012 .008	.037 .013 .013 .009 .012 .015 .006
J. O. Handy, Mgr. Research Dept., Pgh. Testing Laboratory.	.110	.094	.024	.021
J. O. Handy, Mgr. Research Dept., Pgh. Testing Laboratory.	.088	"Scale-free" butt weld— .075 Reg. .089	.0132	"Scale-free" butt weld— .0111 Reg. .0134

each piece under test.
Harvard University, where 1¼" pipe was used.

conditions to a different set of conditions to which they do not and cannot apply.

Some time ago data regarding the durability of wrought iron and steel were obtained, mainly from laboratory corrosion tests which had not been fully confirmed by service tests. During the last ten years, however, a number of service tests have been made to destruction in domestic hot-water lines by various investigators, results of which are briefly summarized in Table 1. It will be seen, from the large amount of evidence which is condensed in this table, that there is no marked difference under these conditions between wrought iron and steel. These tests were all conducted by connecting together these materials in the same service line so that they were all subject to the same conditions.

To avoid drawing erroneous conclusions in the future, and to assist in the general study of corrosion, the following classification of the different types of corrosion has been suggested. This is based on the different external conditions under which corrosion is known to occur.

1. Atmospheric corrosion.
2. Corrosion under water.
3. Soil corrosion (underground).
4. Chemical corrosion.
5. Electrolysis due to imposed electric currents.

This discussion is confined exclusively to corrosion under water, where the metal is continuously in contact with the water. Corrosion under these conditions is dependent upon the transfer of oxygen through water to the surface of the metal. Oxygen is only soluble in water under normal conditions, to the extent of about eight to ten parts per million, so that the damage which a certain volume of water may do is limited by the small amount of dissolved oxygen *in closed systems.*

Considering the means which may be employed for protecting pipe from corrosion by water by improvements in the method of manufacture, from what has just been said it will be seen that there is a limit in the variation of composition within which very little difference has been found in the rate of corrosion. However, if large amounts of chromium, silicon, or nickel are added, the rate of corrosion in water is materially decreased to about $^1/_{10}$ of the normal rate. These ferrous alloys are, unfortunately, expensive, and cannot be welded, so the pipe manufacturer must now use soft steel or wrought iron, and is circumscribed by certain fixed limitations in the composition, on account of the necessity of welding, threading, and fabricating the material. Nevertheless, much has been done to improve pipe material during recent years. A process has been developed, which is now in commerical use in a number of mills, for the removal of the mill scale formed in welding. This scale is the magnetic oxide of iron and is largely responsible for localizing corrosion and starting pits.

Other preventive measures consist in the application of protective coatings to the pipe. The most permanent of these consists of cement

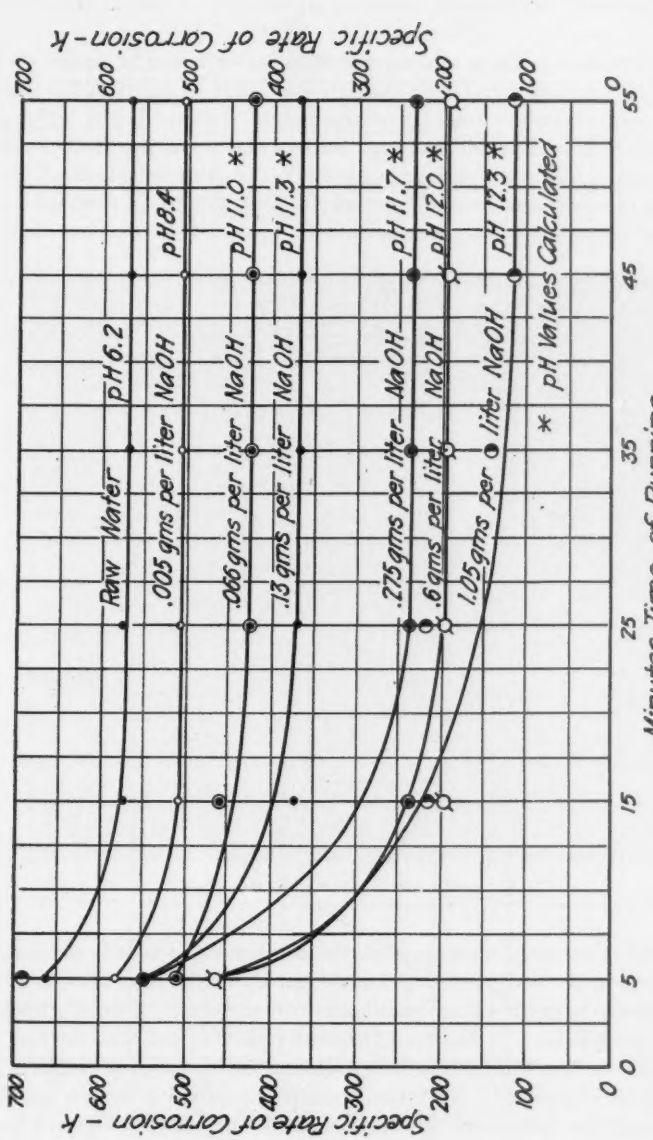

FIG. 1. — SPECIFIC RATE OF CORROSION WITH VARIOUS CONCENTRATIONS OF SODIUM HYDROXIDE. ¼ INCH STEEL PIPE, 150° F., PITTSBURGH CITY WATER.

and lead lining. Protective coatings may also be applied naturally by treating the water with certain materials which tend to form a protective scale on the pipe.

The difference between natural waters in the corrosion of pipe is very marked, and has been found to be due almost entirely to differences in their power to form protective coatings on the metal. The soft water which is so common in New England has very little of this tendency, so that corrosion is very active in this district; whereas, in the cities using Great Lakes water, the pipe will last three or four times as long, due to the formation of

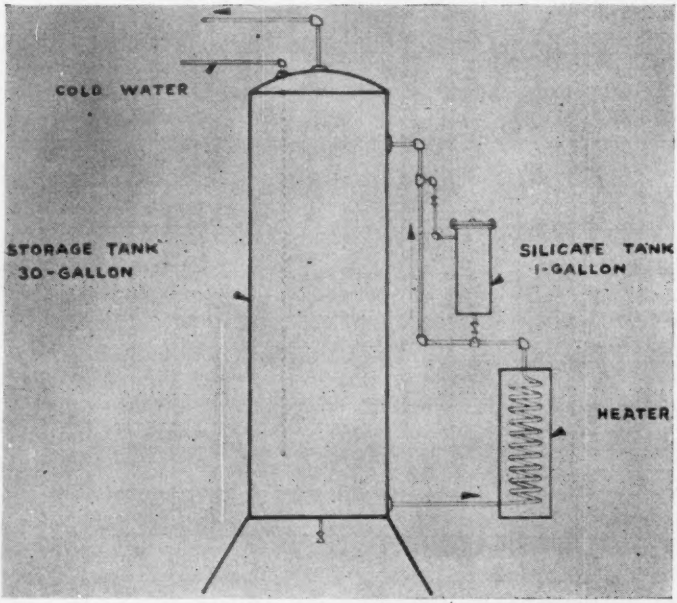

FIG. 2. — SILICATE TANK FOR SMALL SYSTEM.

substantial protective coatings from the substances present in the water. These natural protective coatings consist mainly of the rust formed in the initial corrosion of the metal, combined with carbonate of lime, silicates, and organic matter. It has been frequently pointed out that the rate of corrosion in certain localities in New England has increased since filtration plants were introduced. Filtration naturally removes a certain amount of the important scale-forming constituents of the water. Figure 1 indicates the changes which occur in the initial corrosion of iron with water which is neutral, and with water which carries certain amounts of alkalinity. It will be seen that the rate of corrosion to start with is almost the same, but alkaline water becomes much less corrosive after a short time, due to the

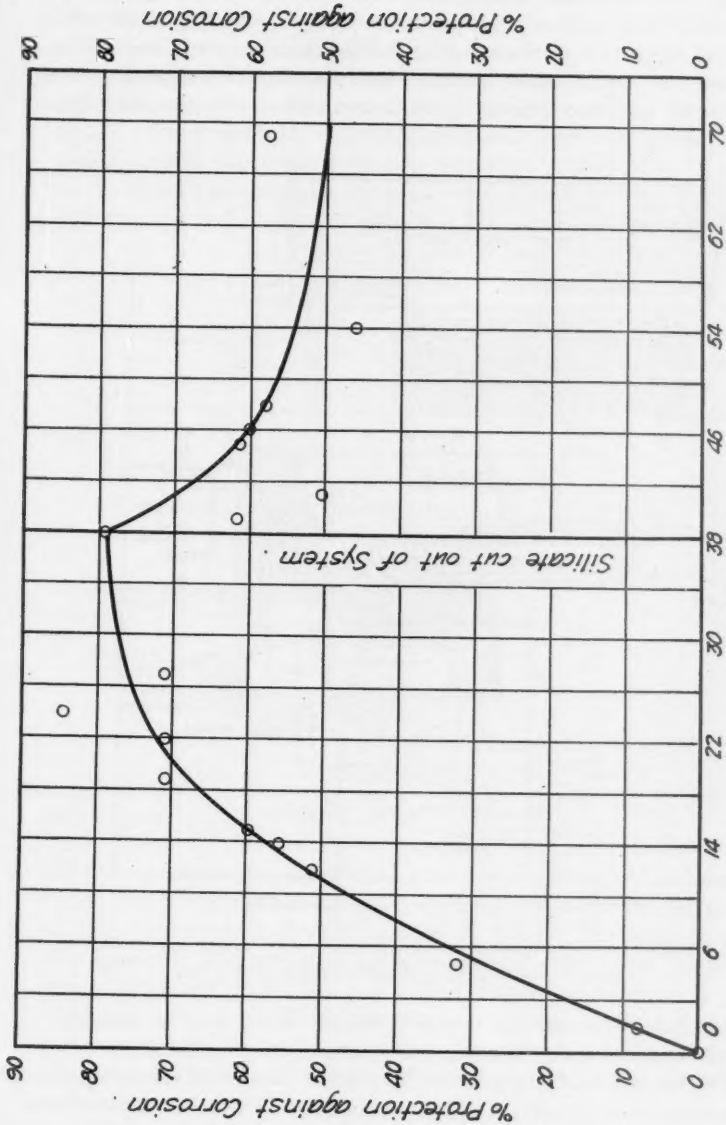

FIG. 3. — LAUNDRY SILICATE INSTALLATION. 18 p.p.m. SILICATE ALKALINITY. 140 CU. FT. WATER PER DAY at 130° F.

formation of a protective film consisting mainly of the insoluble hydroxides of iron. This desirable property of water may be obtained by the addition of about one-half grain per gallon of lime, or sufficient to neutralize nearly all aggressive carbonic acid. Under this treatment, some calcium carbonate will be deposited, if any of the bicarbonates are present. The same result may be obtained by the use of sodium silicate. All natural

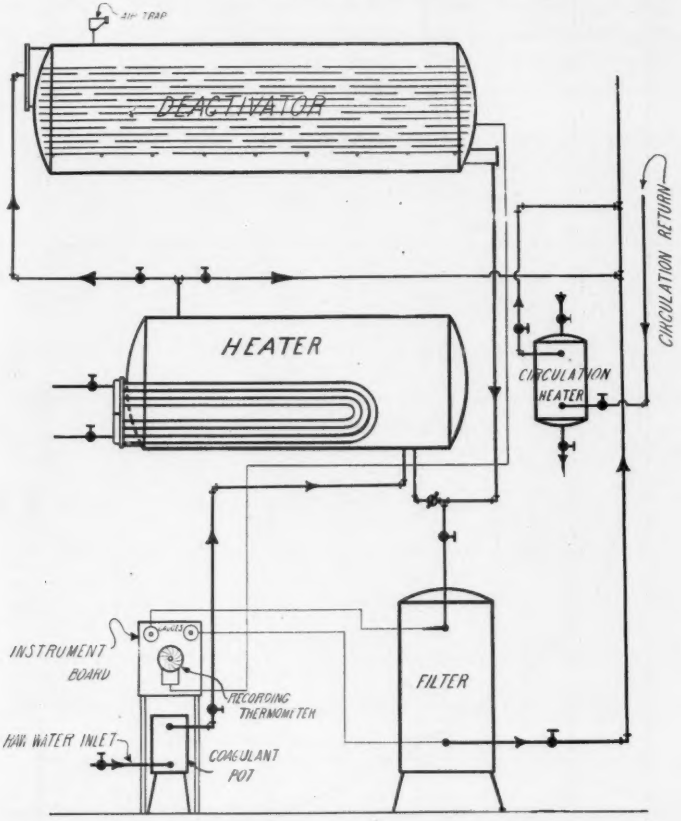

FIG. 4. — MECHANICAL DEACTIVATOR AND FILTER.

waters carry small amounts of soluble silicates which seem to have an important influence in the formation of natural protective coatings.

Silicates are particularly useful in eliminating the red-water nuisance, and may be easily applied to the hot-water system by the use of anhydrous silicate of sodium in an auxiliary tank attached to the hot-water boiler, as shown in Fig. 2. The use of this treatment has resulted in a reduction of eighty per cent. in the rate of corrosion, as determined by tests at the Massachusetts Institute of Technology on Cambridge water, and at the Mellon

Institute of Industrial Research on Pittsburgh water (Fig. 3). Sodium silicate has been used effectively in England on a large scale for the treatment of city water supplies, where it is said to have given good results. The addition of this compound has the advantage that it does not harden the water, although it is slightly more expensive than lime. This treatment does not need to be continuous, as the coating formed will last for a few days to a few weeks without further additions, after a continuous scale has been deposited, depending upon the character of the water.

In closed water systems, corrosion can be almost entirely eliminated by removing the free oxygen in solution. The actual amount present is so

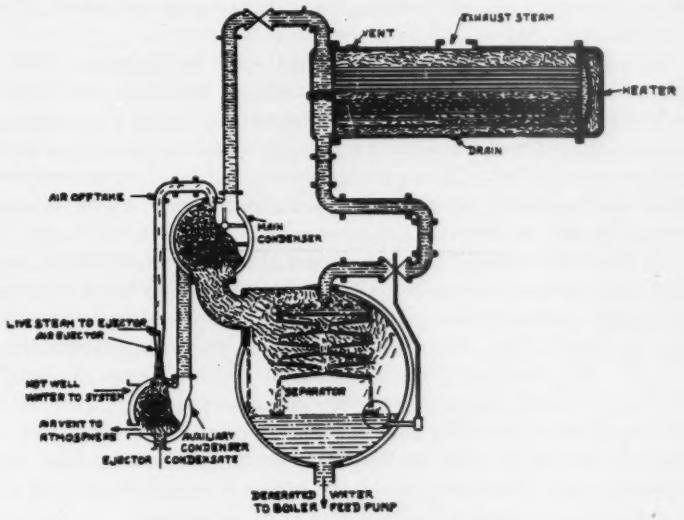

FIG. 5. — ELLIOTT DEAERATOR.

small that this has been found to be an entirely practical form of treatment which may be carried out either by chemical or mechanical means. The so-called deactivation of water consists in keeping the water in contact with expanded steel sheets, or an open form of steel scrap at a temperature of about 160° F. for one hour. This will usually fix the oxygen, after which the water is practically inactive towards iron or other metals. To secure perfectly clear water, mechanical filtration is sometimes necessary. This process may be used economically with a rate of flow up to two thousand gallons per hour (Fig. 4).

Above this point, it has been found more economical to remove the oxygen mechanically. This may be done by boiling the water at atmospheric pressure and reducing the temperature by means of a heat exchanger, if desirable. Another process for mechanical de-aeration is illustrated in

Fig. 5, in which the water is heated and passed into a chamber under vacuum, so that it boils violently and discharges a large amount of steam which carries with it the dissolved gases. The vapor is drawn by means of a vacuum pump or steam ejector, through the condensor by means of which the condensable water vapor is separated and returned to the system. In this way, almost complete de-aeration can be obtained in handling very large volumes of water.

Cold water can be de-aerated in much the same way, but not to the same extent, by using a vacuum of over twenty-five inches of mercury at normal temperature. A plant of this kind was installed two years ago in the power plant of the Christian Science Benevolent Association in Boston. This has been operating quite satisfactorily in removing eighty-five per cent. of the dissolved oxygen.

This has been attempted on a larger scale in connection with the Coolgardie Water Supply (Western Australia). The most recent reports indicate that by treating the water in a high vacuum at 80° F., about eighty per cent. of the oxygen is removed, which has considerably reduced the rate of corrosion in the water mains of this system. It is, of course, important to keep the de-aerated water from contact with air, as after such treatment oxygen is readily re-absorbed.

In closing, I would like to add a word as to the necessity for closer co-operation in corrosion research. This work has been done heretofore almost entirely by individual initiative, with the financial aid of large corporations. The time seems to have arrived when the work should be coördinated and enlarged. The National Research Council has appointed a committee consisting of the chairmen of corrosion committees of the various National Engineering Societies. The main object is to prevent overlapping of these investigations, and to make the results of this committee work more helpful to all concerned. Standard units of the corrosion measurement, and standard methods of laboratory testing should be established. So far, it has been very difficult to compare the work of one investigator with that of another, on account of the variations in the methods of testing and the units of measurement. It would seem that an Association such as this one should be identified with constructive work of this kind.

PROCEEDINGS.

January Meeting.

City Club, Boston,
Tuesday, January 13, 1925.

The President, Stephen H. Taylor, in the chair.

Morris Knowles presented a memoir of Michael F. Collins.

Frank A. McInnes presented a memoir of Frederic I. Winslow.

An illustrated talk on Corrosion was given by Frank N. Speller, Metallurgical Engineer, National Tube Company, Pittsburgh, Penn. William A. MacKenzie, A. O. Doane, and Robert Spurr Weston took part in the discussion.

A discussion on Progress and Experiences of Forestation of Watersheds, some costs, figures, and description of methods, was opened with papers by Theodore N. Bristol and Edward E. Minor, followed by a discussion led by Harold O. Cook, Chief Forester, State of Massachusetts, and participated in by George A. King, Percy R. Sanders, Elliot R. B. Allardice, Samuel E. Killam, F. N. Strickland, Henry T. Gidley, Edward D. Eldredge, and William A. MacKenzie.

(*Adjourned.*)

February Meeting.

City Club, Boston,
Tuesday, February 10, 1925.

The President, Stephen H. Taylor, in the chair.

President Taylor announced that the next convention would be held during the week beginning Sunday, September 6, 1925, and outlined the trip to be taken.

A paper on The Electric Pumping Plant of the Kennebec (Maine) Water District was read by Arthur L. Shaw, Designing Engineer, Metcalf & Eddy.

A paper was read by Burt B. Hodgman, National Water Main Cleaning Company, on Some Results of Pipe Cleaning.

Desmond Fitzgerald, Caleb M. Saville, Charles W. Sherman, Frank J. Gifford, Arthur E. Blackmer, S. H. MacKenzie, and President Taylor took part in the discussion.

A topical discussion on Coöperation Between Fire and Water Departments was opened by the reading of a paper by Thomas E. Lally, Assistant Engineer, Water Service, Public Works Department, Boston.

John S. Caldwell, Chief Engineer, New England Insurance Exchange, also read a paper on this subject.

Edward A. Barrier, Patrick J. Hurley, Patrick Gear, Harry A. Burnham, Herbert C. Crowell, Woodbury Hale, J. Harry Holmes, William F. Sullivan, W. G. Classon, Henry T. Gidley, David A. Heffernan, William H. Hill, Frank A. McInnis, Richard H. Ellis, C. D. Sharpe, and George A. King took part in the discussion.

(*Adjourned.*)

March Meeting.

City Club, Boston,
Tuesday, March 10, 1925.

The President, Stephen H. Taylor, in the chair.

Secretary Gifford. The Executive Committee at this morning's meeting voted to hold the June outing at Providence, R. I. We were there some five years ago to start things, and Mr. Winsor assures us that at this time the construction of the dam and other work are at a very interesting stage. The June outing will be about the 25th. This is only tentative, and the actual date will be announced later.

President Taylor. I have a letter here which I think the Association would like to hear:

"A. Betant
Chief Ingenieur
of the water-supply
Geneva. (Switzerland)

Geneva, 21–II, 1925.

Stephen H. Taylor, Esq.,
President of the N. E. W. W. A.
Boston, U. S. A.

Dear Sir:

I have been very sorry to read in the Journal of the N. E. W. W. A. that we have lost our colleague Prof. G. C. Whipple.

I can't forget that it is here, during his stay in Geneva, that I have had the pleasure to become acquainted with Professor Whipple and that it is thanks to him I have had the honour to be admitted as a member of the N. E. W. W. A.

Therefore I hold it a duty to express to you most sincerely my deepest regrets for the loss our Association has made.

Yours faithfully,
(Signed) A. Betant."

You will remember Mr. Betant prepared a paper a year ago, which was read before the Association by Mr. Whipple. It is a very proper tribute to our late member.

A paper on The Diesel Engine and its Overall Efficiency, illustrated by stereopticon charts, was read by C. B. Jahnke, Chief Engineer, Fairbanks-Morse & Company.

A talk on Diesel Engine Experience in a Pumping Plant was given by Leonard C. L. Smith, Consulting Engineer, Long Island City.

A paper on Diesel Engine Experience in a Modern Forge Plant was read by J. P. Harbeson, Jr., Chief Engineer, Camden Forge Company, Camden, N. J.

The general discussion on the Diesel engine was participated in by Samuel A. Agnew, Leonard Metcalf, R. H. Bacon, and J. Howard Hayes.

Papers on Protection of Water Pipes and Appurtenances in Exposed Locations were read by Samuel E. Killam, Superintendent of Distribution, Metropolitan District Commission, and George H. Finneran, Superintendent of Water Service, Boston. Charles W. Sherman, A. O. Doane, George F. Merrill, Edward D. Eldredge, Frank J. Gifford, Melville A. Sinclair, John Doyle, and President Taylor took part in the discussion.

PRESIDENT TAYLOR. I want to take this occasion to thank our three guests who have come today and given us these papers. They are very interesting and will make a great addition to our JOURNAL.

(*Adjourned.*)

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Dear Sir: Enclosed please find 1.50 in payment of charge for Certificate
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Yours truly,

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Specializing in Water Works
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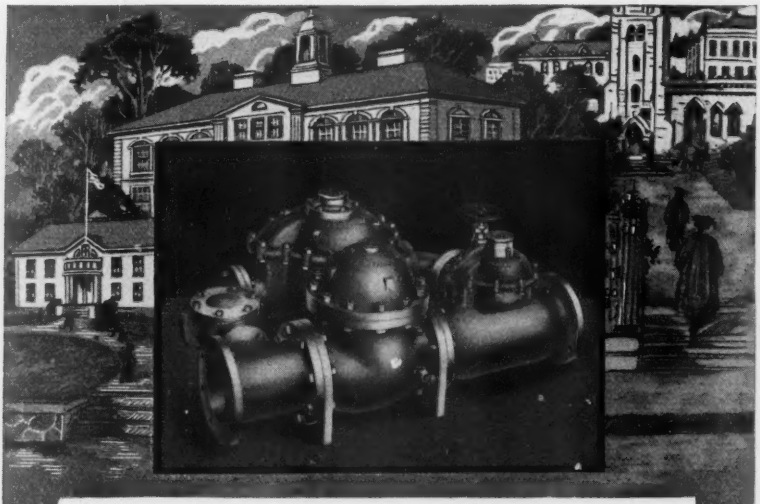
Telephone, Richmond 1200



287,804 SCHOOL BUILDINGS

are worth less than half the value of the property this Meter protects

THE HERSEY DETECTOR METER is installed on fire services protecting nearly $5,000,000,000 worth of property, or a sum over 50% in excess of the value of the 287,804 school buildings in the United States.

Not theory but the practical experience of 20 years has gained the unrestricted acceptance of the Hersey Detector Meter for every kind of fire service by every insurance company, stock and mutual, in the country and more than 800 water departments and water companies.

This record has no equal.

HERSEY WATER METERS

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PORTLAND, ORE. ..... 63 Front Street
CHICAGO, ILL. ..... 10 So. La Salle Street
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FROZEN!—but not damaged

Here is an actual photograph of a LAMBERT Frost-proof Meter which has been frozen to an extent that would put the ordinary meter completely out of commission.

This is made possible by a patented, non-corrosive yielding bolt device which allows the upper and lower casing, disc chamber and gear train to part without damaging the meter in any way. Five minutes' labor the only repair cost.

The expense and annoyance from frozen water meters can be eliminated for all time through the installation of the LAMBERT Frost-proof.

It has been proved that the LAMBERT is the easiest water meter to take apart and put together again as well as the simplest, most reliable and accurate.

If you are interested in other types of meters, we make one for every requirement.

THOMSON METER COMPANY

100-110 BRIDGE STREET BROOKLYN, N. Y.

LAMBERT FROST-PROOF METERS

Metering Solves Price City's Water Problem

UNTIL 1919, Price City, Utah, depended upon the badly polluted Price River for its water. In 1920 a pipeline was laid through a difficult mountainous region to bring pure water to the town from Colton Springs, twenty-five miles away.

During the first year all service was on a flat rate basis, and in a short time the pipeline was running at full capacity. During 1921 the department had to put restrictions into effect to insure consumers an adequate supply for essential uses. Recognizing the necessity for a measured service, a meter program was begun. The city is now 100 per cent metered.

These are some of the results which metering has accomplished by eliminating the wasting of water:

Every consumer has had an ample supply of water, even during the severe drought of 1924 when the flow was about half of the normal.

Railroads, coal camps and other towns have become customers of Price City for pure Colton Spring water.

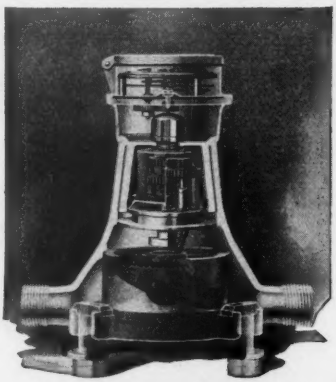
Price City consumers are enjoying absolutely pure water at costs which are less than formerly under the flat rate.

Receipts of the water department have increased from $5,567 in 1919 to $33,904 in 1923.

Regardless of the size of a city, metering will eliminate water waste, and conserve this valuable resource, without restricting the legitimate use of water. Metering distributes the cost of water in the most equitable way and reduces the municipal cost of supplying water — thus benefiting every inhabitant.

Investigate the results where Neptune Trident Meters have been installed.

AQUAFAX is a 56-page, monthly magazine giving current news and accurate information regarding the conservation of water. The regular subscription price is $3.00 per year, but it will be sent free upon request to engineers, municipal heads and civic organizations.



TRIDENT Meters are built to give a service which for accuracy, durability, and low maintenance costs, cannot be rivaled. Due to the unit system of construction, old Trident Meters now in service can be modernized — and new Trident Meters are practically impregnable against the elements of deterioration.

There are more than 2,750,000 Trident Meters in use.

Send for a copy of our new 86-page book illustrating and describing all types and sizes of Neptune Trident Meters.

NEPTUNE METER COMPANY

Pioneers in Meter Progress

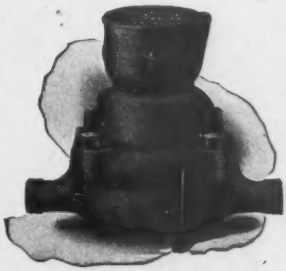
50 EAST FORTY-SECOND STREET, NEW YORK CITY

Neptune Meter Co., Ltd., 1197 King Street, West, Toronto, Ont.

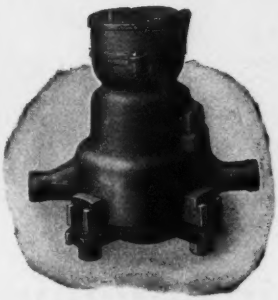
Boston Chicago San Francisco Atlanta Los Angeles Seattle Portland St. Louis, Mo.

WATER METERS

The TROPIC — An all-bronze Water Meter, with connection spuds attached to the lower case. Particularly designed for warm sections of the country.



TROPIC



ARCTIC

The ARCTIC — A frost-bottom meter, especially designed for cold sections of the country.

These two meters embody exactly the same mechanical features, the only difference being the changes necessary to provide a frost breaking feature in the Arctic.

Either of these meters will be equipped with a COMPLETELY ENCLOSED intermediate train RUNNING IN OIL, if desired.

Write or wire nearest office for full information.

Please Note Change of Address

PITTSBURGH METER COMPANY

7800 Susquehanna St., Pittsburgh, Pa.

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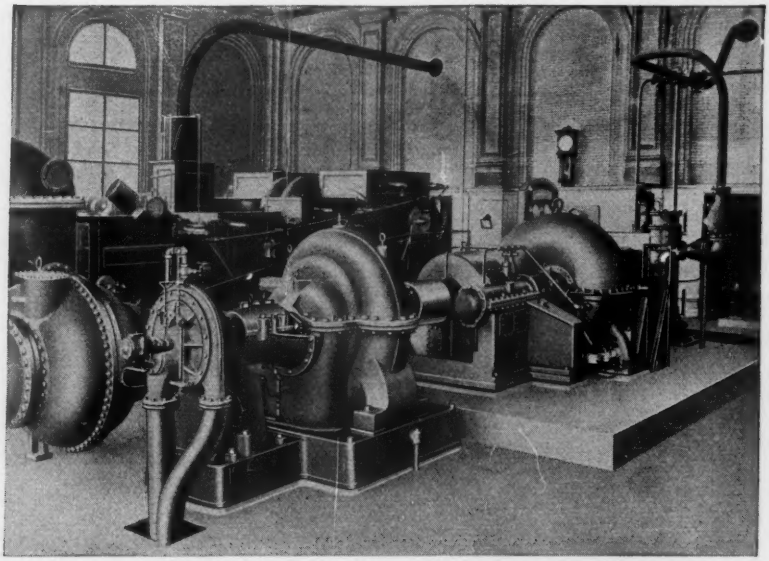
Los Angeles, Union Bank Building



Mount Royal Pumping Station of Baltimore, Maryland

Centrifugal pump efficiency 86.2 per cent

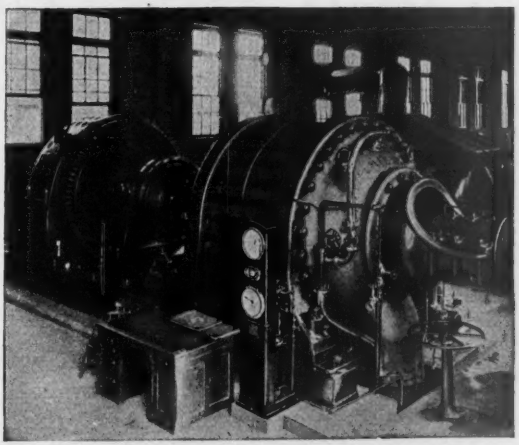
WORTHINGTON engineers are not contented to follow. They strive continuously for leadership. The talent which has made the name Worthington respected by waterworks engineers continues its success, giving municipalities of the world the benefit of over *seventy* years' accumulated experience.

An illustration of Worthington soundness of design and construction is the 30,000,000 gallon centrifugal pumping unit in the Mt. Royal Pumping Station of Baltimore, Maryland. This highly economical unit is driven by a G. E. Steam Turbine exhausting into a Worthington Surface Condenser equipped with Worthington auxiliaries. The complete installation is described in a bulletin which will be sent to anyone upon request.

WORTHINGTON PUMP AND MACHINERY CORPORATION
115 BROADWAY, NEW YORK BRANCH OFFICES IN 24 LARGE CITIES

WORTHINGTON

AFTER THE PUMP IS ACCEPTED



Pumping Station of Montreal Water & Power Co.

Pump performance is bound to deteriorate as the months go by.

Just when are repairs needed to prevent the original investment from becoming highly costly?

Progressive towns and cities answer this question positively by installing a VENTURI METER in the discharge line from each pump.

Bulletins No. 83 and 194 on Pump Performance are available

BUILDERS IRON FOUNDRY

Builders of the Venturi for 34 Years

PROVIDENCE, R. I.

VENTURI

"VENTURI" Registered

AMERICAN AND NIAGARA WATER METERS

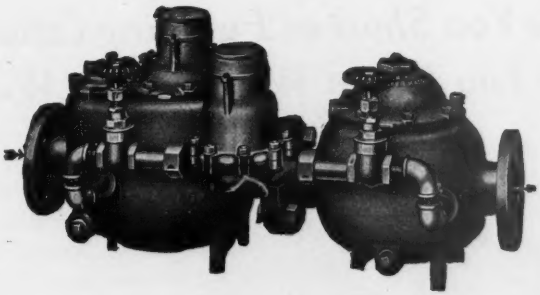
American and Niagara meters are made in solid casing types and frost bottom types. The illustration above shows a $^5/_8$" x $^3/_4$" meter of the frost bottom type. Those meters having bronze outside casings are known as Americans and those having galvanized cast-iron outside casings are Niagaras. The works in all the different casings are of one best model and interchangeable. The registers may be either round reading or straight reading and may indicate cubic feet, U. S. gallons, imperial gallons or litres. All Niagara and American meters are tested and adjusted according to the requirements of the Standard Specifications for water meters adopted by the New England Water Works Association and the American Water Works Association.

Write for large catalog and prices.

BUFFALO METER CO.

ESTABLISHED 1892

2896 Main Street **BUFFALO, N. Y.**



The New Nilo

The new improved **Nilo Compound Meter** now comes equipped with a bronze body. This is a valuable addition to meter service for it eliminates the danger of corrosion and filling up due to the collecting of corrosive matter. Bronze being stronger than iron the gross weight of the meter is less, making it easier to transport, handle and install.

It also has two gate valves which allow the removal of the by-pass meter for cleaning or repairing without shutting off the line.

The controlling valve mechanism of the new **Nilo Compound Meter** is self contained and easily detached from the main body casings.

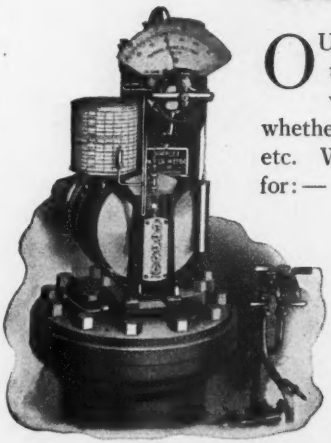
These three improvements make the **Nilo**, a long standing favorite, the unrivaled leader in its field.

UNION WATER METER CO.

Incorporated 1868

WORCESTER, MASSACHUSETTS

Is Your Pressure Low?
Are You Short on Pumping Capacity?
Do You Know Where Your Water Goes?



OUR Engineering Department will tell you how to determine your water distribution, how to find out whether or not new mains are required, etc. We can supply you with the means for: —

1. Checking up pump performance and determining slippage due to leaky plungers, defective valves, short stroking or other causes,
2. Measuring delivery of centrifugal pumps and figuring whether or not the station is up to efficiency,
3. Distributing proportionately the cost of water supplied to several districts,
4. Checking up the performance of filter beds,
5. Detecting waste or pipe leakage,
6. Recording daily amounts of sewage handled and planning future extensions, etc., etc.

THE SIMPLEX METER

VENTURI, PITOT TUBE OR ORIFICE TYPES

THE importance of exact measurements by means of a Simplex Meter cannot be over emphasized. Mr. Jeffries, Chairman of the West Chester, Pa. Water Works Committee, was able, by checking the performance of his pumping engine with a Simplex Meter, to discover that due to the imperfection and wear in the valves the slippage amounted to 300,000 gallons of water per day. This represented a daily loss of $30.00, or in other words a daily saving of $30.00, or $900.00 per month, when the valves were put in order. The Simplex Meter thus paid for itself in a month's time.

Frequent repeat orders from satisfied customers who formerly used other meters are the best evidence of merit of the Simplex Meter.

We have had many years' experience in solving water flow problems involving all sorts of conditions, and we are at all times prepared to submit general layouts involving cost and capacity, and to make explicit and complete recommendations. Write for Bulletin N26.

SIMPLEX VALVE AND METER COMPANY

Manufacturers of Meters for Water, Sewage, and Other Liquids, Rate Controllers, Automatic Air Valves, Regulating Valves, and Hydraulic Apparatus of Special Design.

5729 RACE STREET PHILADELPHIA, PA.

"WATCH DOG" WATER METERS

ACCURATE *DURABLE*

DISC, CURRENT AND COMPOUND TYPES

Inquiries Solicited

GAMON METER COMPANY

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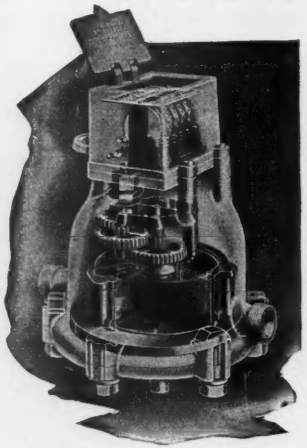
WATER METERS



The EMPIRE has been on the market for forty years, and in all that time no other meter has ever equaled it for accuracy and endurance. That isn't a bad record to look into if the meter-buyer wants to get the most for his money; for when all's said and done it is the close registering meter that brings in the revenue, and it is the long wearing meter that keeps down expenses. A customer who recently gave us a nice order for EMPIRES told us that he did so because he had had about forty-five of them in service for sixteen years — all testing today up to standard — and in that whole time they had *never cost one cent for repairs.*

WHAT EMPIRES HAVE DONE FOR OTHERS

THEY CAN DO FOR YOU

Don't take our bare word for it, — let us send you names of some towns using EMPIRE meters extensively, so you can do your own investigating.

EMPIRES are made in all sizes 5/8″ to 6″.

NATIONAL METER COMPANY

299 BROADWAY, NEW YORK

Chicago Boston Cincinnati Atlanta San Francisco Los Angeles

There is a complete line of Badger Water Meters from the small 5/8" disc type to the large 6" Turbine Compound. You will find a type and size Badger for every service.

The Proven Water Meter

You can conscientiously specify Badger Meters and be assured that they will perform dependably and accurately day after day. They have been tried, tested and have proven themselves. That is why they are being used in ever increasing numbers, the country over.

Badger Meters give satisfactory service because they are carefully constructed. They are intended to give long life, and they do.

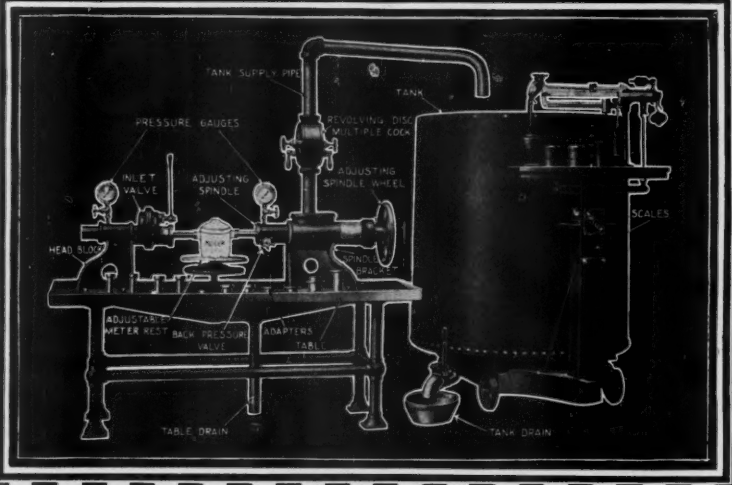
We are always glad to render advice on meter systems and stand ever ready to demonstrate the superiority of Badger Meters — theoretically, structurally, mechanically and practically. Let us send you special circulars giving detailed information.

Badger Meter Mfg. Co., 853 30th St., Milwaukee, Wis.

111 W. Washington Street, Chicago, Ill.
414 Interstate Bldg., Kansas City, Mo.
32 Court Street, Brooklyn, N. Y.
1621-39 Fifteenth Street, Denver, Colo.

BADGER METERS

"Faithful to the last drop"



The People Demand Increased Efficiency

You can make sure your Department gets full revenue for every gallon of water used—by seeing that all meters are regularly tested for accuracy—with the

Mueller Water Meter Tester

The Mueller will prove exactly what each meter does—will enable you to adjust each meter to absolute accuracy—will save more than its cost year after year. Used and endorsed by leading municipalities. The name Mueller guarantees both Quality and Service.

Detailed description and prices on request.

MUELLER CO., Decatur, Illinois
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Water, Plumbing and Gas Brass Goods

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ATTENTION
WATER WORKS
OF NEW ENGLAND

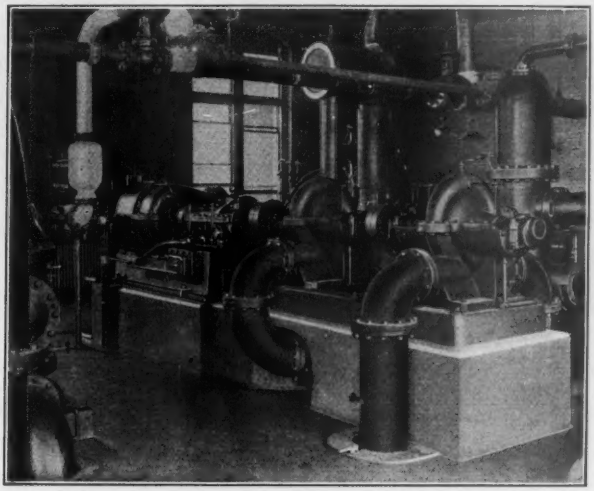
A few reasons why we solicit your inquiries on Water-Works Pumping Equipment:

First. — We make a specialty of furnishing Complete Water-Works Pumping Units of all types.

Second. — Total responsibility of installation is assumed by us.

Third. — We relieve the purchaser of every detail and turn the complete unit over to him under actual operation and with guarantee.

Fourth. — Every unit installed to date has exceeded its guarantee and has been completely accepted.



Turbine-driven Pump at the Arlington Station of the Metropolitan Water Works, Boston, Mass.

F. A. Mazzur & Co.

141 MILK STREET, BOSTON

Centrifugal Water Works Pumps

BUILT FOR THE LAST 70 YEARS AT BALDWINSVILLE, N.Y.

By Morris Machine Works

Reliable **Substantial** **Efficient**

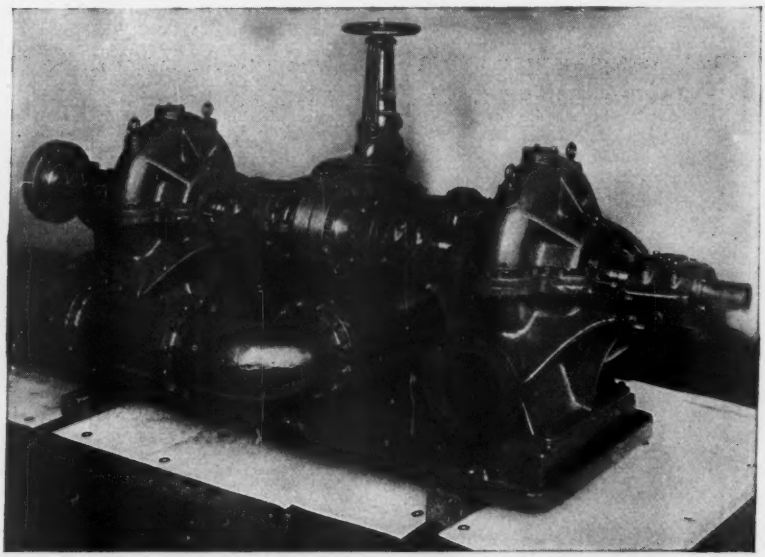


Illustration shows two 3 M.G.D. pumps just furnished as part of complete waterworks equipment to Kennebec Water District, Waterville, Me., to plans and specifications of Metcalf & Eddy, Consulting Engineers, Boston. These pumps will be driven by direct connected Motors and with Gasoline Engines for stand-by service. Arrangement permits single or series operation depending upon head, which varies greatly with capacity required.

We furnish pumps for all heads and capacities, motor turbine or engine driven, and when desired install complete units ready for operation.

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"SERVICE BEFORE PROFIT"

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Pumping Plant Contractors

CENTRIFUGAL, POWER, STEAM PUMPS

STEAM TURBINES
WATER WHEELS

ELECTRIC MOTORS
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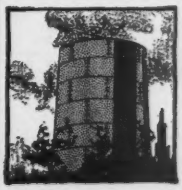
Power Plant Equipment
Steam and Heating Specialties

For new pumping equipment guaranteed by people who will make good, phone, wire or write the above.
Sales Agents for UNION STEAM PUMP CO.



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Water Tanks
Gas Holders
and all other metal surfaces need the protection of
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JOSEPH DIXON CRUCIBLE CO. Jersey City, N. J.

WE announce our appointment as Sales Representatives of DAYTON-DOWD COMPANY, of Quincy, Ill., Manufacturers of Centrifugal Pumping Machinery. After exhaustive investigation, we present this line of equipment to the Trade in New England as being of the highest grade and thoroughly modern in design and manufacture.

We are prepared to furnish DAYTON-DOWD pumps in all sizes and for all classes of service, in steam, motor, gasoline and belt drive types, and solicit your inquiries, assuring you of our best efforts in your sales and service requirements, and of our appreciation of your continued confidence and patronage.

POWER EQUIPMENT CO.

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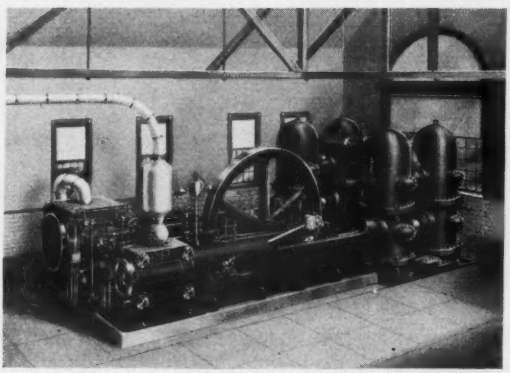
MURRAY IRON WORKS CO.

(INCORPORATED 1870)

BURLINGTON, IOWA, U. S. A.

Builders of High Duty Pumping Engines



Murray High Duty Crank and Fly Wheel Pumping Engine, Opposed Type

Engineers, Iron Founders, Boiler Makers

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Water-Tube, Fire-Tube and Internal-Furnace Boilers

Builders of Complete Power Plants

PLEASE SEND US YOUR SPECIFICATIONS AND CALLS FOR BIDS

When The Snow Melts!

SPRING Floods will come—excessive pollution will be washed into water supplies—and increased vigilance will be necessary to prevent the spread of Water Borne Disease.

For nearly four thousand communities in North America sterilizing every drop of drinking water W & T Apparatus will stand guard against this inevitable spring time pollution.

Is your town insured against water borne disease?

"The only safe Water is a Sterilized Water"

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Manufacturers of Chlorine Control Apparatus
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FILTER ALUM

Aluminum Sulphate

A basic, uniform product which has been used successfully by the leading water filtration plants in New England for many years.

Our location and shipping facilities place us in a position to offer insurpassable service and make us the logical source of supply for all filter plants in New England.

The aid and advice of our technical staff has often proved of great value to our customers. It is always at their service.

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Works:

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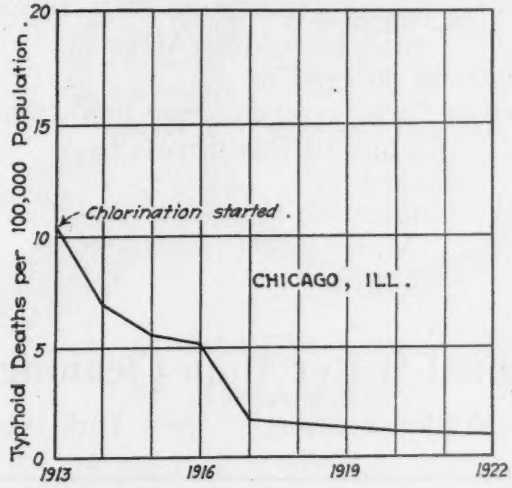

DIAGRAM SHOWS REDUCTION IN TYPHOID FEVER DEATH RATE AFTER INSTALLATION OF CHLORINE

Ask for

WHITE MOUNTAIN BRAND LIQUID CHLORINE

LINDER & MEYER

ESTABLISHED 1848

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Selling Agents for

THE BROWN COMPANY

BERLIN, N. H.

Health and Protection First

Water delivered through dirty pipes
may be a MENACE.

Incrusted water pipes mean inefficiency
and loss of Fire Protection.

We Guarantee the Results of Our
Method of Cleaning.

WRITE US.

National Water Main Cleaning Co.

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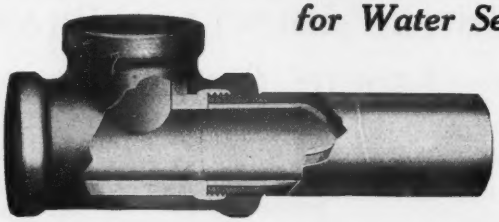
LEAD LINED IRON PIPE AND FITTINGS
CEMENT LINED IRON PIPE
for Water Service



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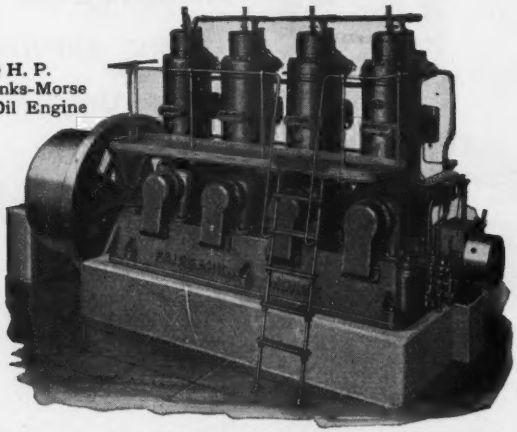

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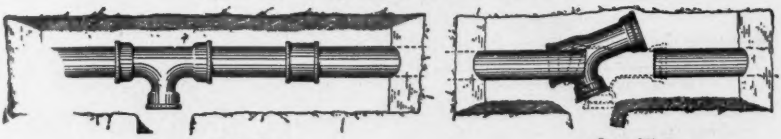

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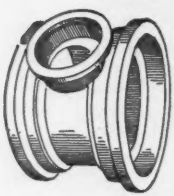

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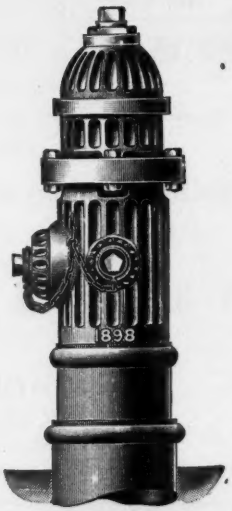

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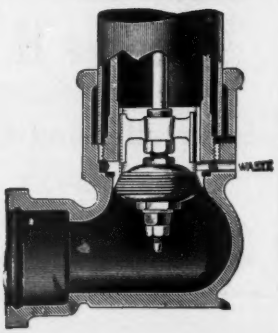

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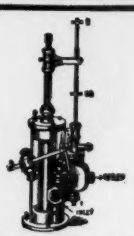

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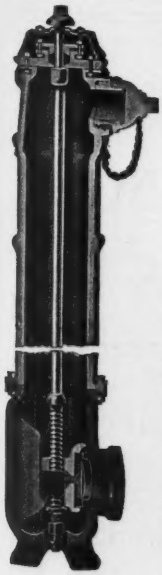

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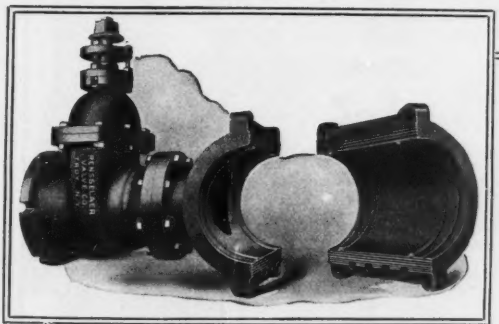

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They can be used with all standard makes of Tapping Machines.

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Have two lead rings.

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Iron washers not required.

Bolting flanges are made full length of sleeve, giving great strength.

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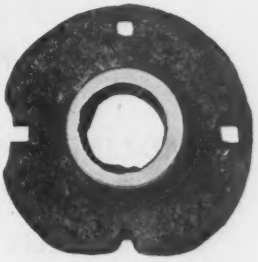

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Lead Lined Iron Pipe Company

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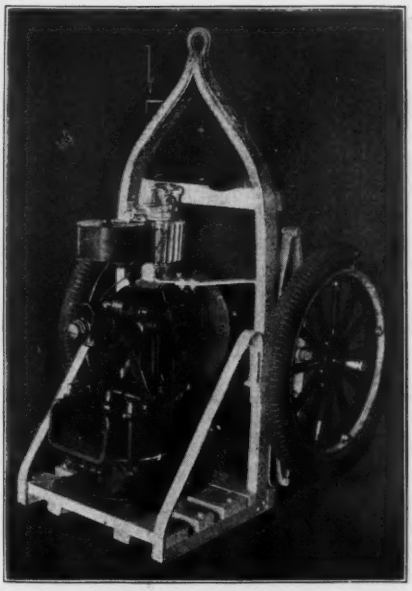

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TEN CENTS PER POUND PRICE TEN CENTS PER POUND PRICE TEN CENTS PER POUND TEN CENTS

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50% FIRST COST

50% SMALLER BELL HOLES

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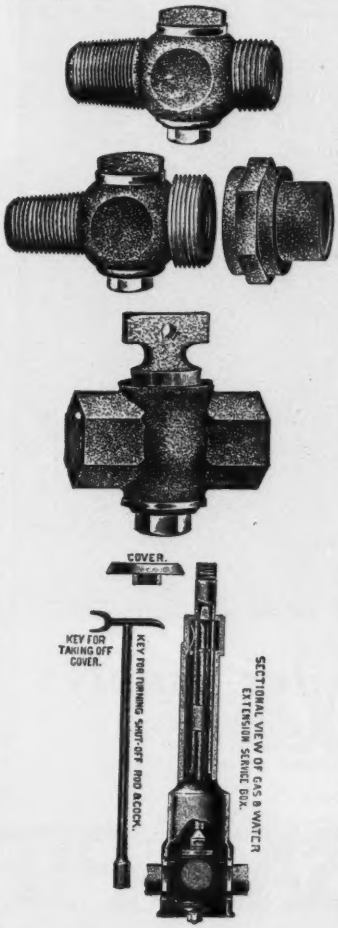

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We have them to meet every requirement

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Extension Service Boxes of proven advantages

Let us send at our risk, on 30 days' trial, a

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which is recognized to be the most easily operated owing to the few working parts.

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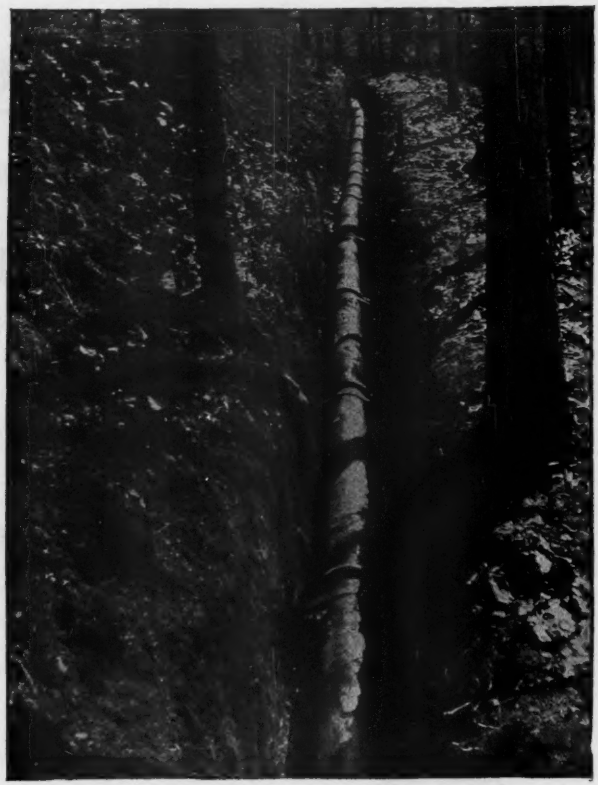

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giving perfect service
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no packing no calking no bell holes

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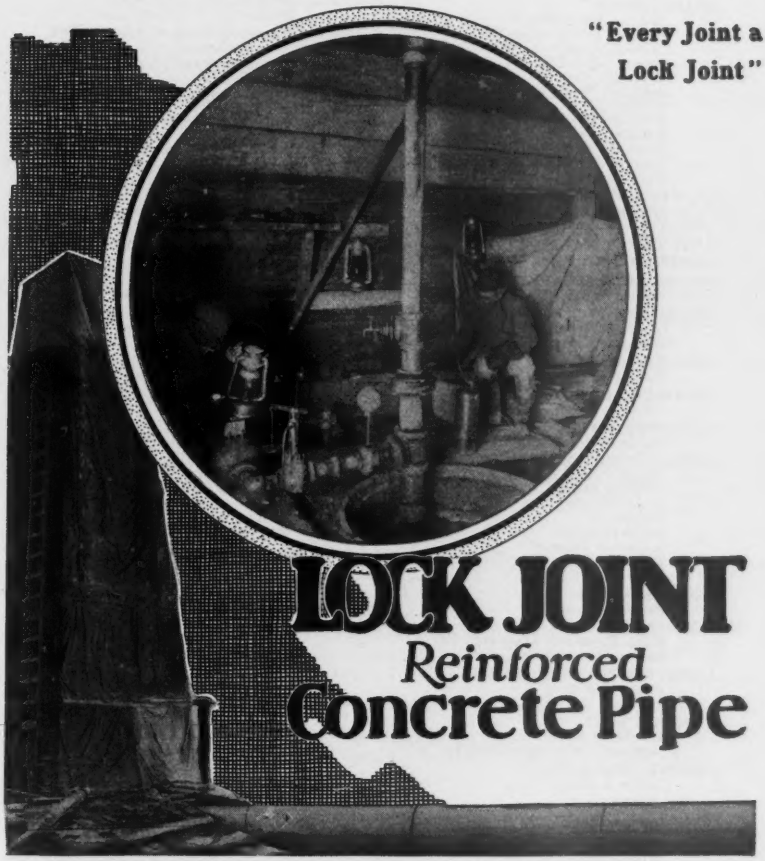

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Pressure, Sewer, Culvert and Subaqueous Pipe

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ORGANIZED JUNE 12, 1882.

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